CHAPTER

5

TABLE OF CONTENTS

DESCRIPTION OF SUPERSTRUCTURE TYPES

The following is a list of superstructure types that are most commonly used:

1. Concrete slab spans (precast or cast-in-place)
    a) Solid
    b) Voided
2. Precast-prestressed concrete girder spans
3. Steel
    a) Rolled Beam Span
    b) Welded Plate Girder Span
        1) Multi-Girder Frame
        2) Girder, Floorbeam, Stringer Frame
4. Steel horizontally curved girder span (plate or box girder)
5. Precast-prestressed concrete box girder span
6. Movable bridges

Slab span bridges are the most common bridge type and are generally used at stream crossings where span requirements are not critical and aesthetics are not a major concern. This is generally the most economical bridge type for bridges to **120 m** in total length and even longer depending on pile lengths. Cast-in-place slab spans have been the norm for on-system bridges while precast slab spans are generally used on parish off-system bridges.

Voided slabs are used where 12 m spans are required with depth limitations. Please note that a method to drain and vent each void to prevent water and methane gas accumulation is required.

The next most common bridge type is the concrete precast prestressed concrete girder span, which is used for stream crossings with span requirements exceeding slab span capability, for grade separation structures, and approaches to high level structures.

Rolled beams are used primarily in rehabilitation projects. For new construction, rolled beam spans are seldom used, except for spans where depth limitations are insufficient or inefficient for prestressed girder spans.

Steel plate girder spans are used for longer spans such as river crossings or grade separation structures that require span lengths in excess of **40 m**. Steel box girders are occasionally used in urban areas for aesthetics. The box girder's effectiveness in resisting torsion makes it suitable for horizontally curved girders with long or tightly curved spans. However, I-girders are preferable unless box girders are necessary.

In urban areas where an elevated roadway is required, the precast concrete box girder may be considered in special cases where aesthetics and right-of-way restrictions are critical.

Movable bridge types are discussed in Chapter 12.

BRIDGE RAILING

HISTORY

Since 1928 AASHTO has provided various specifications to address the design and details of bridge railings.

Understanding the evolution of these specifications over the many years is instrumental to the comprehension of the specifications in AASHTO as they appear today.

The reason for a dramatic change in bridge railing specifications has been the need to adapt to the changes in the auto-industry and the wide variety of vehicles, which are present on our highways. In the sixties, AASHTO defined the primary purpose of bridge railing as the ability to **contain** the **average** vehicle. The application of the 10 kip load was established for the design of such railing and it remained the primary criteria in AASHTO through the eighties.

Multiple fatality truck and school bus accidents involving bridge railing, throughout the nation, focused the bridge engineer's attention on whether the 10 kip load closely represented the real life impact loads. The load indicator walls in the crash test sites suggested that the actual loads are in the range of 30 to 200 kips.

In August 1986, FHWA required the full scale crash testing of all bridge rails that are to be used on the federal aid projects. At the same time AASHTO requested the FHWA to assist them in the development of a new bridge rail specification.

THE 1989 GUIDE SPECIFICATION

In 1989 AASHTO adopted a Guide Specification for Bridge Railing. This specification is intended to be a basis for the design of prototype bridge railings that are to be crash tested, and for the design of one-of-a-kind bridge railing where the cost of crash test program may not be justified. The Guide Specification is based on a multiple performance levels' theory, which basically requires a different rail for a different situation. There exist five (5) primary performance levels in this publication. These performance levels are as follows:

PL1	PL2	PL3	PL4	PL4T
Cars & Pick-up trucks	Single unit trucks	Tractor trailers	18 Wheeler	Tanker trucks
5,400 lbs. & less	18,000 lbs.	50,000 lbs.	80,000 lbs.	$80,000^{+}$ lbs.
(2450 kg) & less	(8165 kg)	(22 680 kg)	(36 285 kg)	($36\ 285^{+}$kg)

The Guide Specification contains criteria based on which appropriate performance level is selected. In addition, this publication specifies the various design loads and their strategic locations on the railing for when an analysis is required.

AASHTO LRFD BRIDGE SPECIFICATIONS

In 1994 AASHTO published its first series of LRFD Specifications, both in English and metric units. There exists great similarity between the LRFD railing specification and the Guide Specification. In fact the performance levels and the design loads have been extracted from the Guide Specification and placed in the LRFD Specifications, with the exception that the LRFD offers step by step design criteria and analysis procedures for various bridge railings.

LOUISIANA PRACTICE

Louisiana's primary bridge rail in recent history has been the New Jersey safety shape made of reinforced concrete and in special cases, steel plates. This particular shape has been successfully crash tested for performance level (PL-2). The most recently developed safety shape is referred to as the F-shape. This shape, although not much different than the Jersey shape, has proven to gain a slight advantage over the Jersey shape in redirecting the 8165 kg (18000 lb.) vehicle. For this reason Louisiana has opted to gradually eliminate the use of the New Jersey shape and adopt the F-shape for use on new projects. The application of the F-Shape shall begin with the metric plans. Another advantage in the adoption of the F-shape is the fact that it is the only safety shape that has been crash tested for PL-3 at 1070 mm (42 inch) height.

SELECTION AND DESIGN CRITERIA

Selection of the performance level for all new projects or major rehabilitation projects such as redecking of an existing bridge shall be in accordance with the 1989 AASHTO Guide Specifications and the latest LRFD Specifications. All bridge rails shall be an approved crash tested rail for the specific performance level. All plan details and designs shall also be in accordance with these specifications. The **810 mm** barrier (F-shape PL-2) is currently the standard rail being used on nearly all bridges. When a PL-3 bridge rail is warranted by the AASHTO Guide Specifications, a 1070 mm F-shape is the preferred rail unless the designer deems necessary to use a different rail. When detailing the slab and its reinforcing steel supporting the bridge rail, the minimum slab thickness shown on the crash tested detail shall apply. If thicker slabs are utilized for other design reasons, the reinforcing steel shown in the crash tested detail may be reduced to that amount which provides the same or a greater ultimate moment capacity as the crash tested detail. Additional reinforcing steel may also be required for reasons other than the crash tested quantity, i.e., when using a wider sidewalk than what is shown on the detail. Under the majority of situations, the "F" shape is the bridge rail of choice, however, there are several other crash tested rails which may be applicable. A side mounted open rail system and a solid concrete vertical wall rail is shown here for a PL-2 application. Open rail systems are particularly useful where sight distance, bridge drainage and aesthetics may be of concern.

Rev. 04/01/99

Although open rail systems are usually more expensive, they can significantly improve visibility and drainage.

Steel plate barriers formed to an F-shape are used on movable bridges with steel grid floor.

Additionally, crash tested combination traffic-pedestrian railing is shown here for both PL-1 and PL-2. In majority of cases a combination railing is applied in design speeds of 75 km/h or less, and unless the percentage of truck traffic is high, a PL-1 rail is suitable. However, just as bridge traffic railing, AASHTO Guide Specification shall be used to arrive at the proper performance level for the combination railing. The PL-1 and PL-2 combination railing shown here have been crash tested with a 205 mm curb and a 1525 mm wide sidewalk. The more desired sidewalk width is 1830 mm, and the expert's opinion is that the wider sidewalk will practically enhance the rail performance, as long as the curb height is not increased. The reinforcing steel shown in these details is adequate to resist the crash loads. However, if the sidewalk width or other configuration change, the reinforcing steel and members supporting the sidewalk shall be designed accordingly.

For information on bridge railing end treatments, impact attenuators, roadway barriers, and temporary barriers see Chapter 11, "Barrier and End Treatment Systems".

Miscellaneous Details

A 15 mm open joint is provided in the concrete barrier every 6 to 10.5 m for expansion and contraction. This joint need not be sealed where open deck drainage is not allowed (it is expected to dam itself with debris). For unusual conditions, the engineer could require 150 mm PVC waterstops.

The gap between adjacent barriers at expansion joints shall not exceed 140 mm maximum unless a sliding armored plate is employed to close off the opening.

Rail transition is an important aspect of design involving engineering judgement. The ends of the bridge rail must be protected with some type of transition such as guardrail or other end treatment. Guardrail should be in accordance with latest Standard Plans GR-200 (M), GR-201(M), and GR-202(M). In cases where new construction ties to existing construction there must be adequate transition between sections of F-shape barrier and Brush Curb Rail used on earlier bridges.

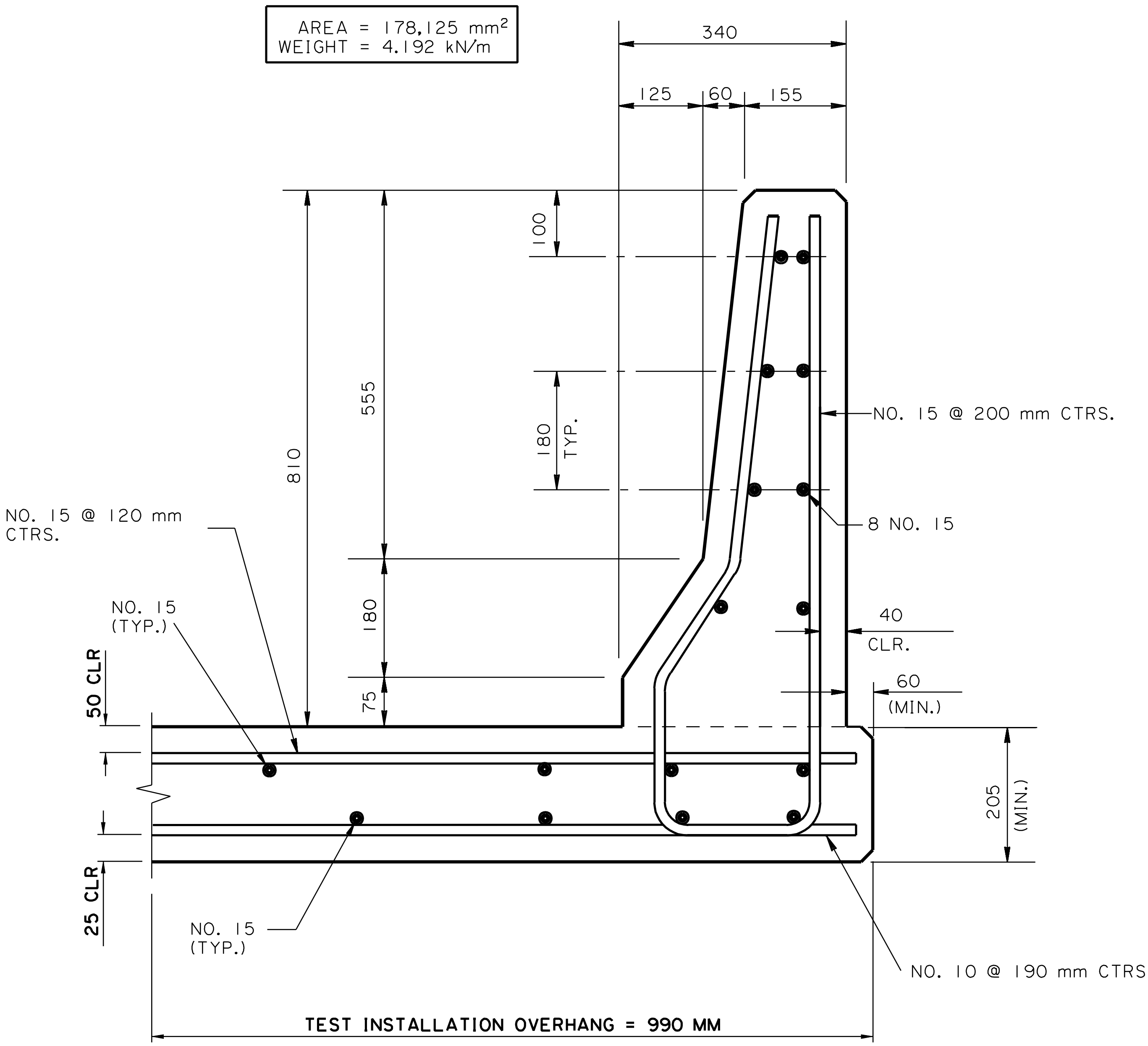
Impact attenuators with back up blocks must be designed for gore areas with oncoming traffic or where typical guardrail can not be used. See the section for crash cushions (impact attenuators) for design.

Bridges in Urban Areas

For bridges with curbed roadway approaches and without sidewalks, F-shape barrier will generally be used with a 1220 mm offset from the edge of travel lane. The roadway curbs will be flared out and discontinued at the guardrail ends. However, in some instances it may be necessary to extend the flared curb behind the guardrail to provide for drainage. This is particularly true when bridge end drains are required.

For bridges with curbed roadway approaches and sidewalks or bikeways, the curb and sidewalk (or bikeway) shall be carried through the bridge. A vertical face parapet with pipe rail is generally used on the outside of the sidewalk, and must meet the requirements for "Combination Rail" mentioned in the AASHTO Bridge Specifications. The guardrail standard will apply and the guardrail shall be placed on the outside of the sidewalk. For higher design speeds, a barrier rail shall be required to separate the sidewalk from the travel lane, and a pedestrian or bicycle rail shall be used on the outside of the sidewalk. The guardrail standard shall apply and the sidewalk will be flared out behind the guardrail.

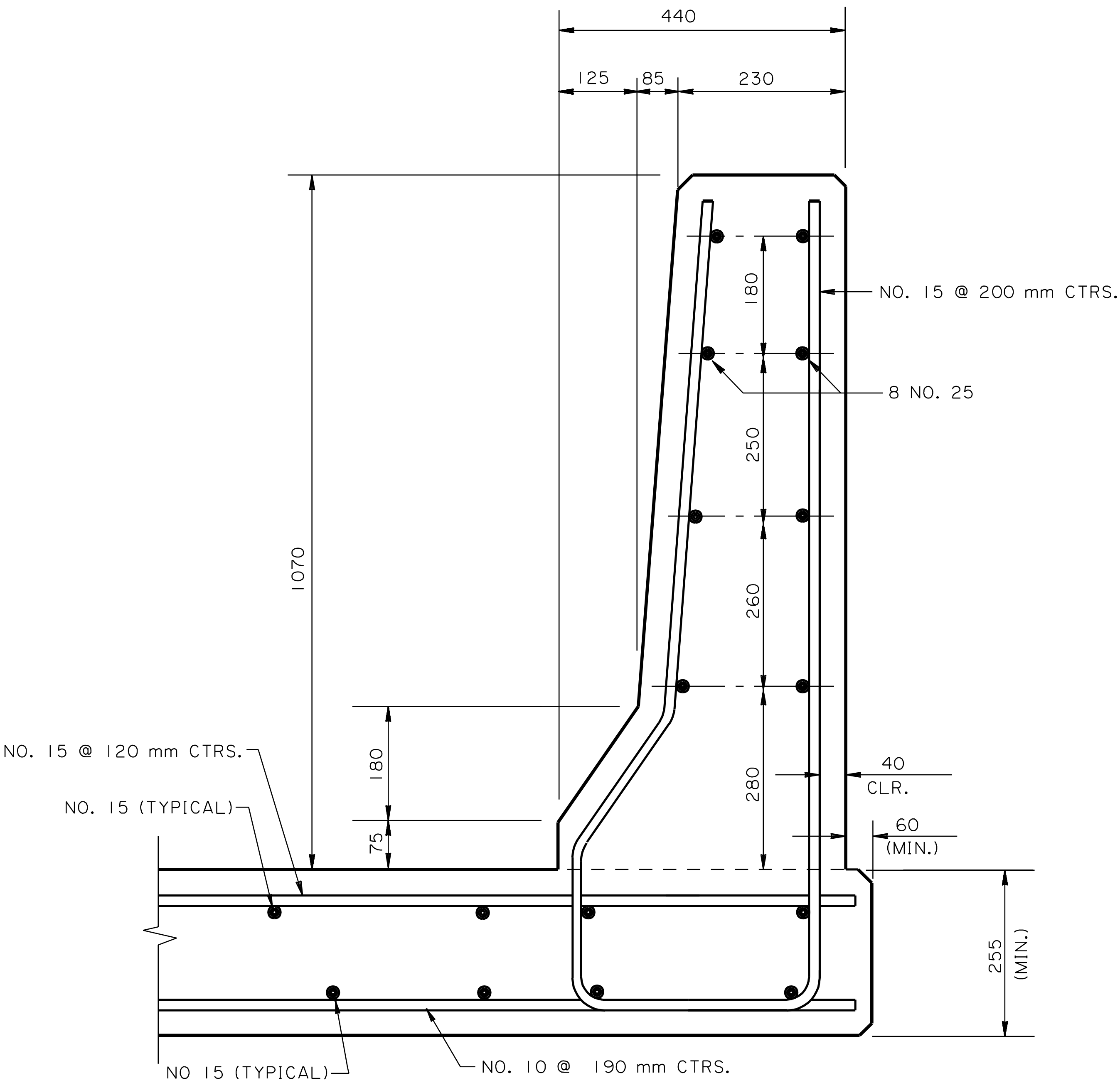
For bridges with design speeds of 70 km/h or less, for barrier end treatment, and sidewalk and curb placement, see EDSM 11.3.1.4.



NOTES:

1) THE ABOVE DETAILS ARE FROM FHWA-RD-93-058 (JUNE 1997) AND HAVE BEEN MODIFIED. SEE 5(4) FOR DESIGN REQUIREMENTS.
2) CONCRETE TO BE CLASS AA
3) REINFORCING STEEL TO BE GRADE 420

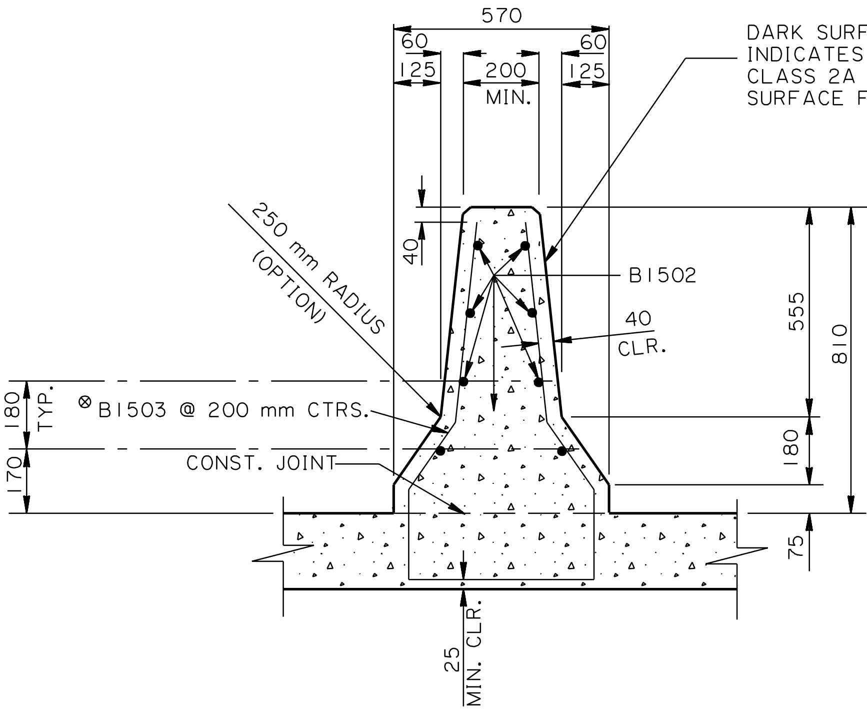
F-SHAPE
(PL-2)



NOTES:

1) THE ABOVE DETAILS ARE FROM FHWA-DR-93-058 (JUNE 1997) AND HAVE BEEN MODIFIED. SEE 5(4) FOR DESIGN REQUIREMENTS.
2) CONCRETE TO BE CLASS AA
3) REINFORCING STEEL TO BE GRADE 400

F-SHAPE
(PL-3)

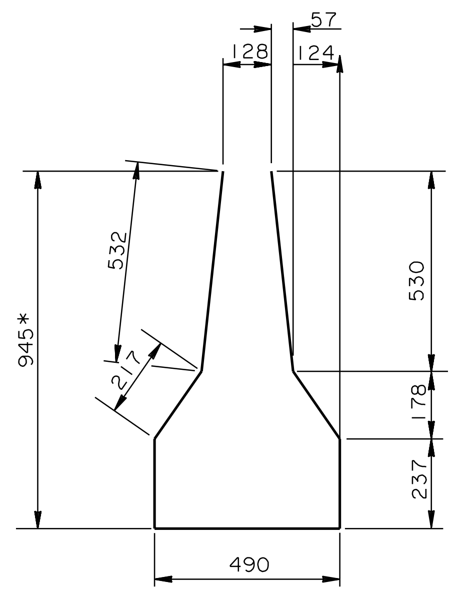


MEDIAN BARRIER (BRIDGE)

⊗ AT THE CONTRACTOR'S OPTION, NO. 10 BARS AT 100 mm CENTERS MAY BE SUBSTITUTED IN LIEU OF B1503 AT 200 mm CENTERS. USE 50 mm Φ PIN FOR BENDING.

* DIMENSIONS SHOWN ARE BASED ON 200 mm THICK DECK UNDER BARRIER; ADJUSTMENTS MUST BE MADE FOR DECKS WITH OTHER THICKNESSES.

AREA - 267 150 mm^2
WEIGHT - 6.288 kN/m

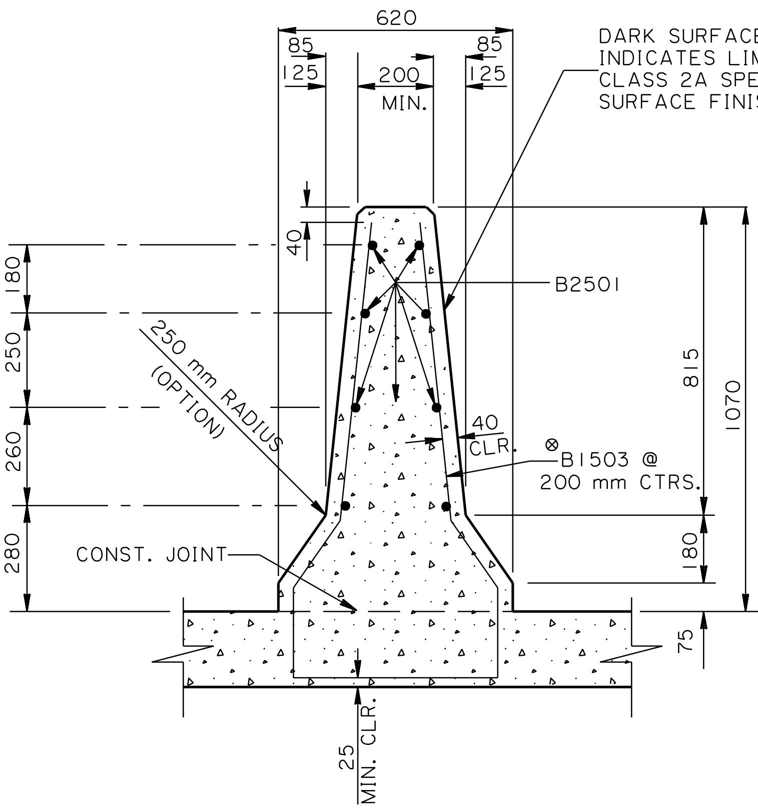


BARS B1503⊗

(60 mm Φ PIN)
(2462 mm LONG)

(F-SHAPE PL-2)

MEDIAN BARRIER (BRIDGE)

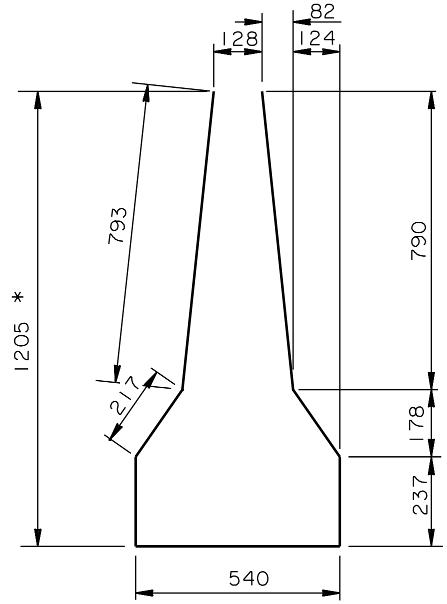


MEDIAN BARRIER (BRIDGE)

⊗ AT THE CONTRACTOR'S OPTION, NO. 10 BARS AT 100 mm CENTERS MAY BE SUBSTITUTED IN LIEU OF B1503 AT 200 mm CENTERS. USE 50 mm Φ PIN FOR BENDING.

* DIMENSIONS SHOWN ARE BASED ON 200 mm THICK DECK UNDER BARRIER; ADJUSTMENTS MUST BE MADE FOR DECKS WITH OTHER THICKNESSES.

AREA - 367 875 mm^2
WEIGHT - 8.829 kN/m

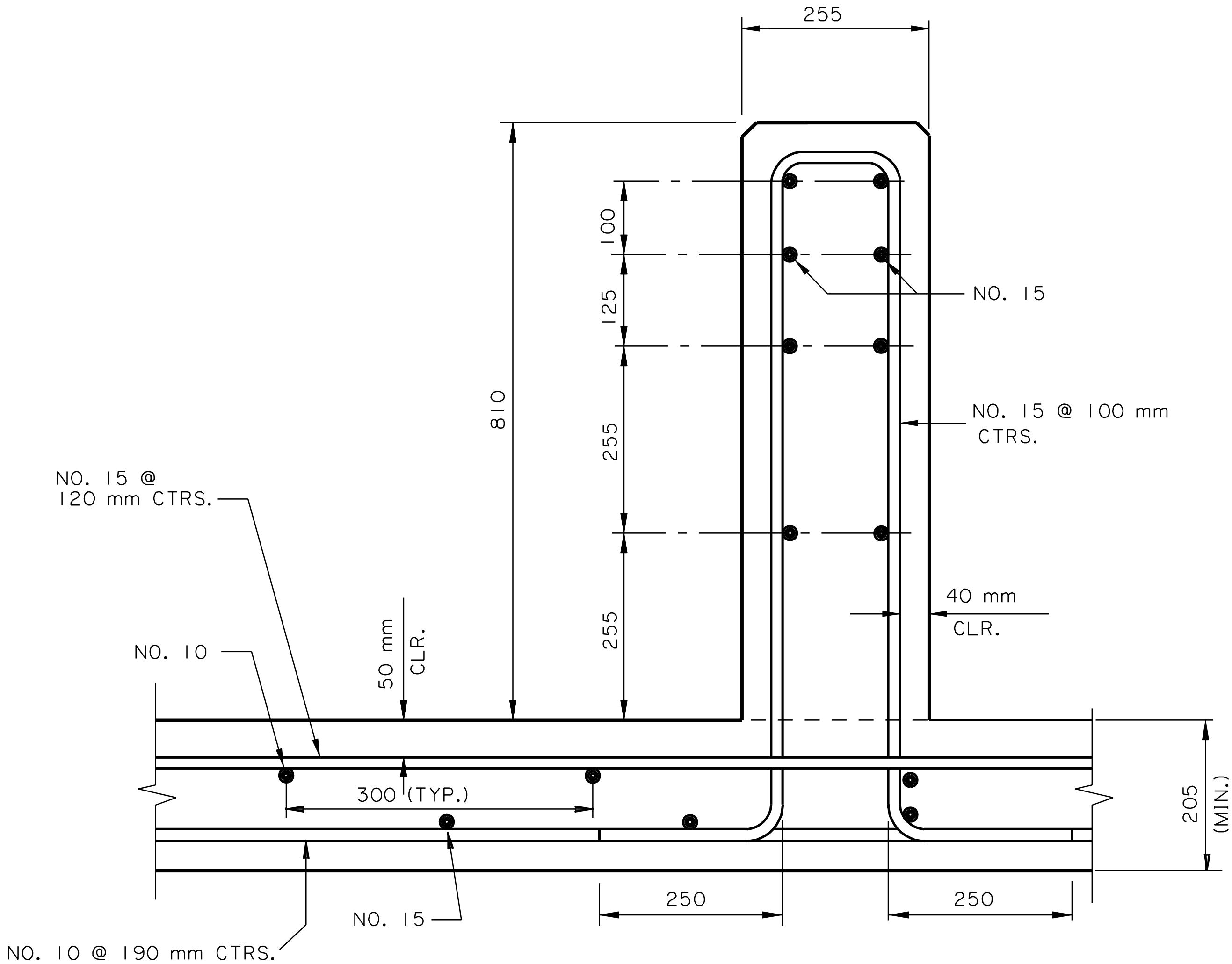


BARS B1503 ⊗

(60 mm Φ PIN)
(3034 mm LONG)

(F-SHAPE PL-3)

MEDIAN BARRIER (BRIDGE)

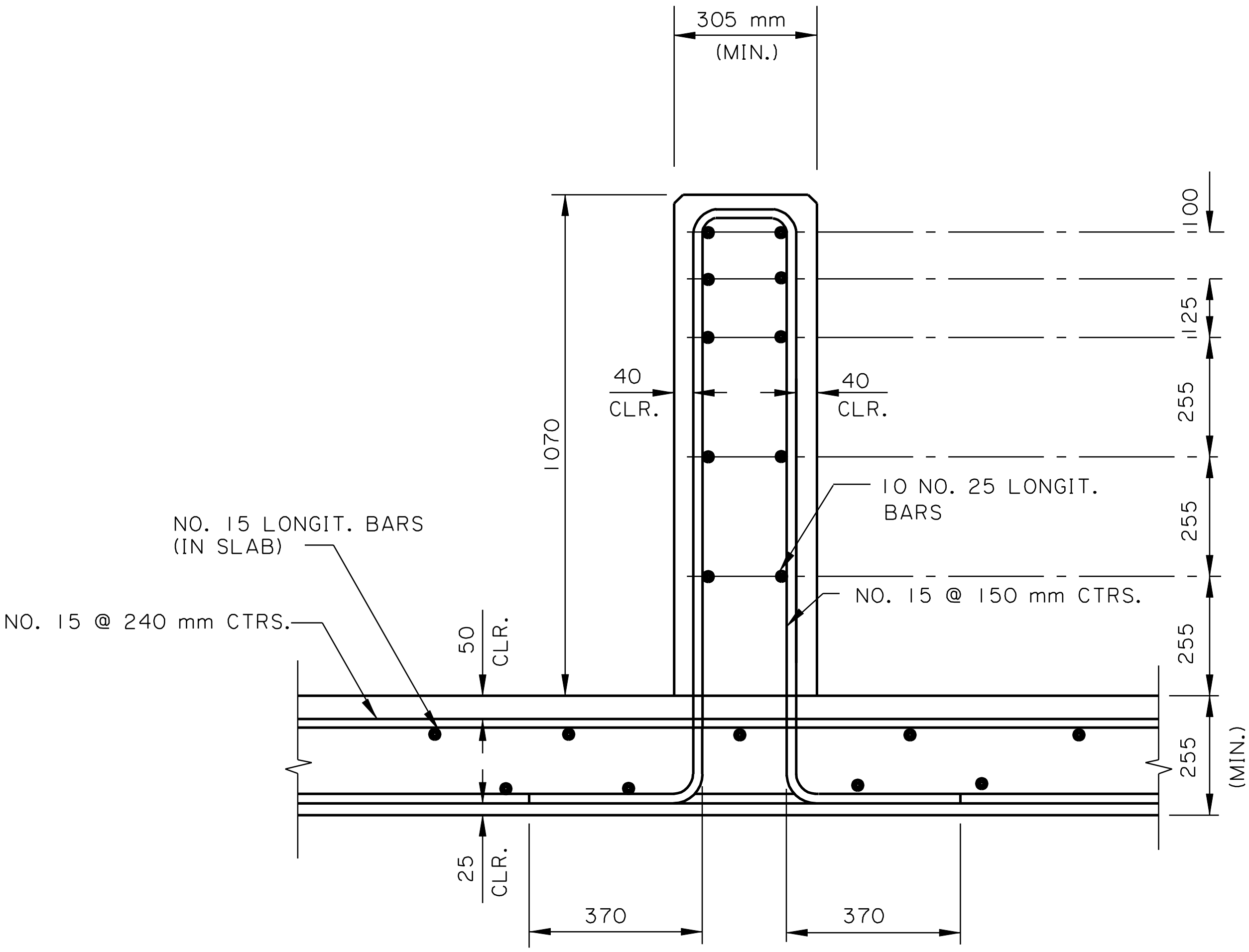


NOTES:

1) CONCRETE TO BE CLASS AA
2) REINFORCING STEEL TO BE GRADE 420

(VERTICAL WALL PL-2)

MEDIAN BARRIER (BRIDGE)

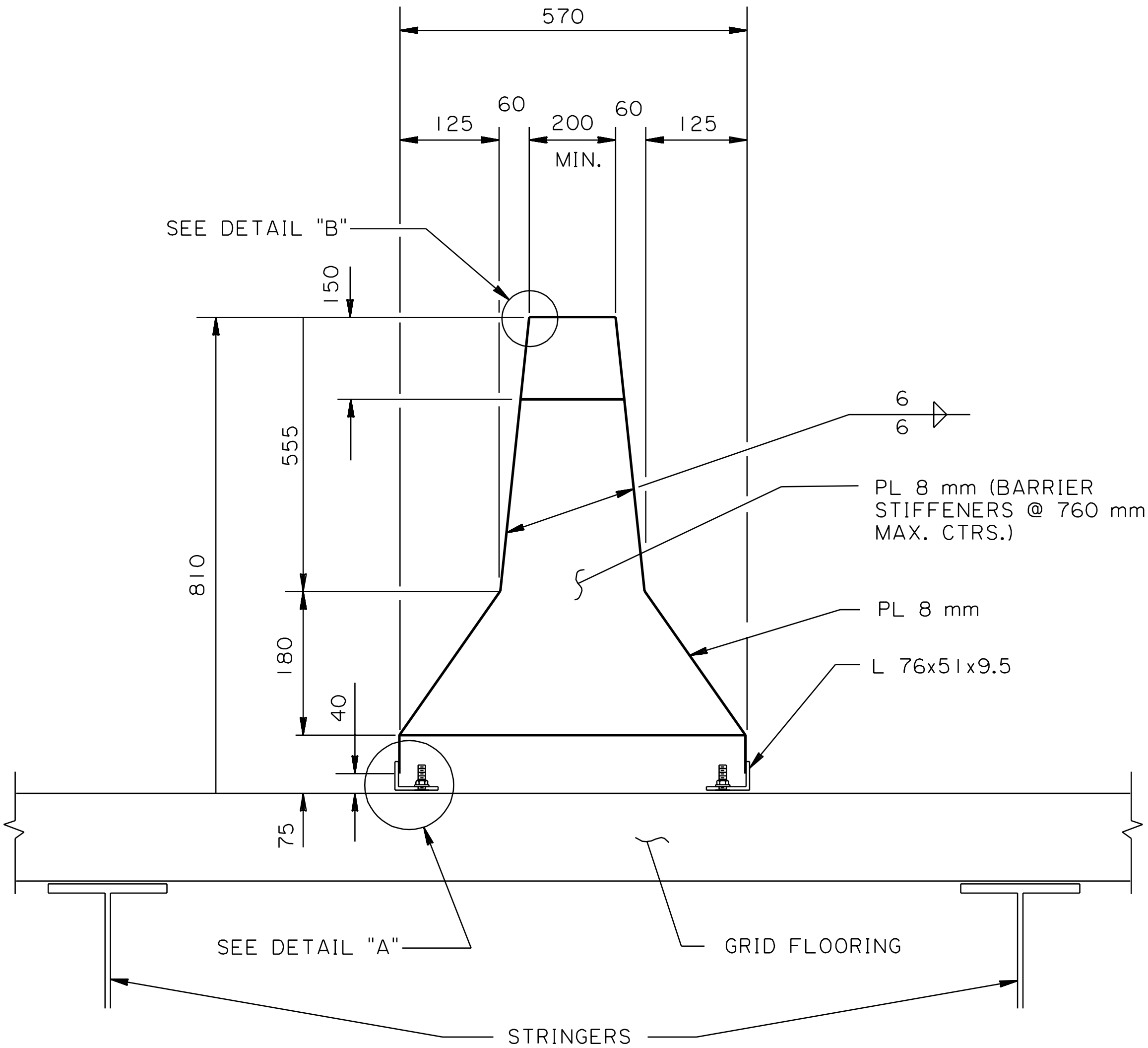


NOTES:

1) CONCRETE TO BE CLASS AA
2) REINFORCING STEEL TO BE GRADE 420

(VERTICAL WALL PL-3)

MEDIAN BARRIER (BRIDGE)

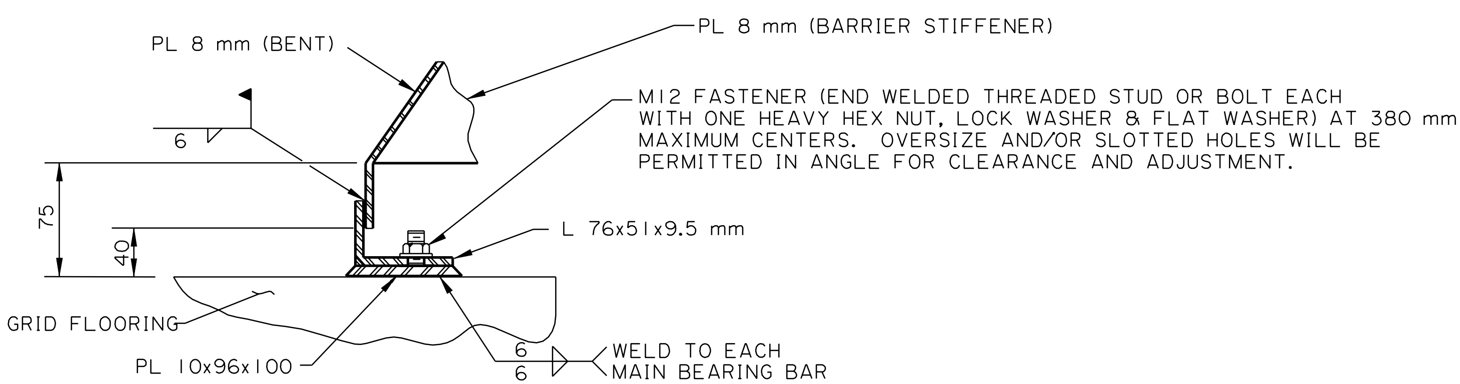


TYPICAL PART SECTION THRU BRIDGE
SHOWING GRID FLOORING AND MEDIAN BARRIER

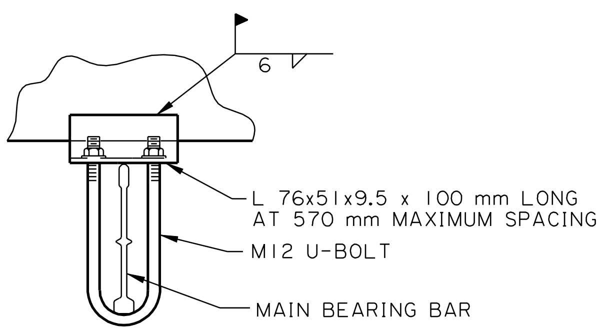
(F-SHAPE)

STEEL MEDIAN BARRIER

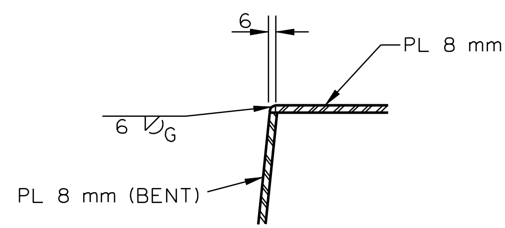
1 OF 2



DETAIL "A"



ALTERNATE

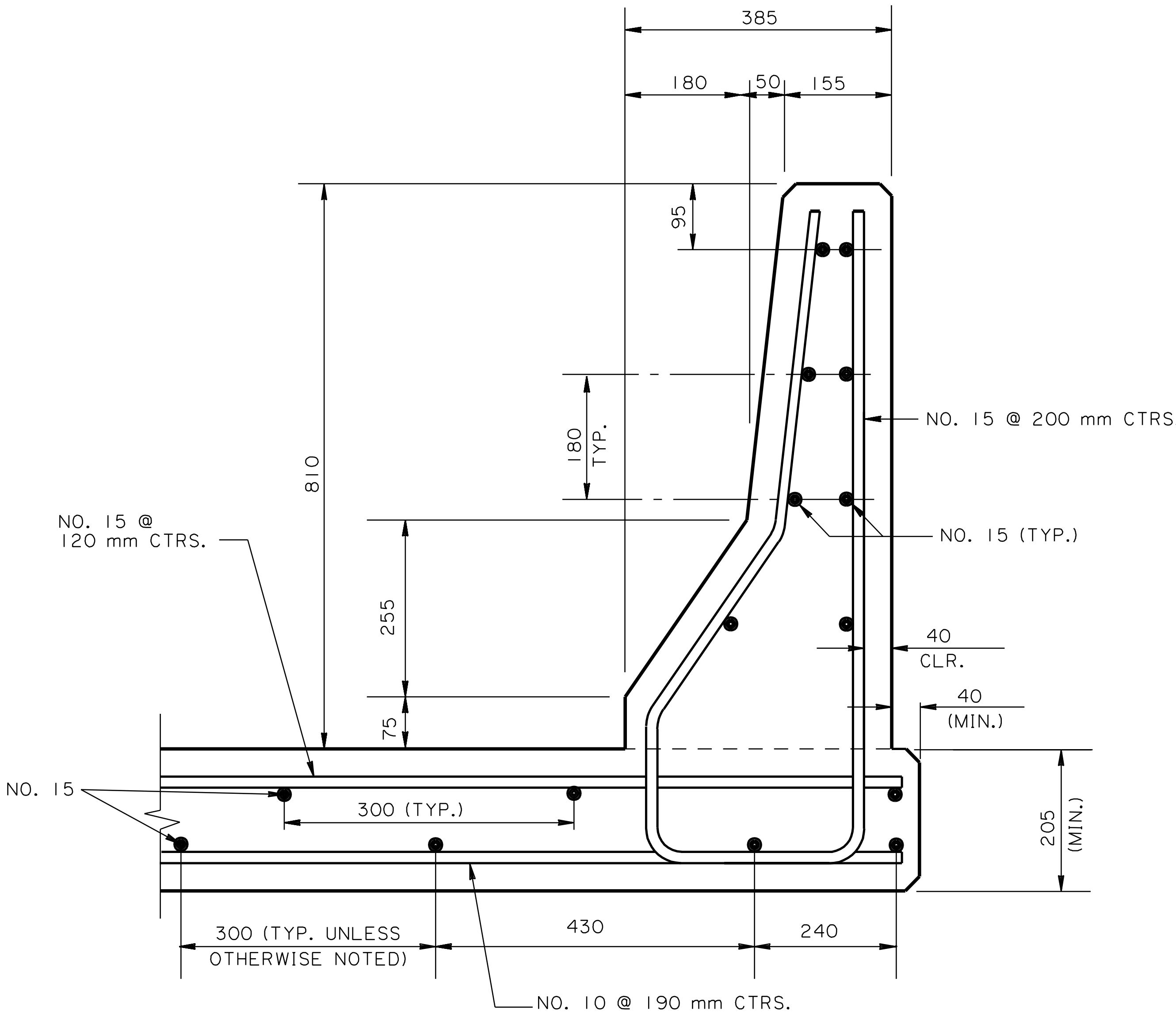


DETAIL "B"

(F-SHAPE)

STEEL MEDIAN BARRIER

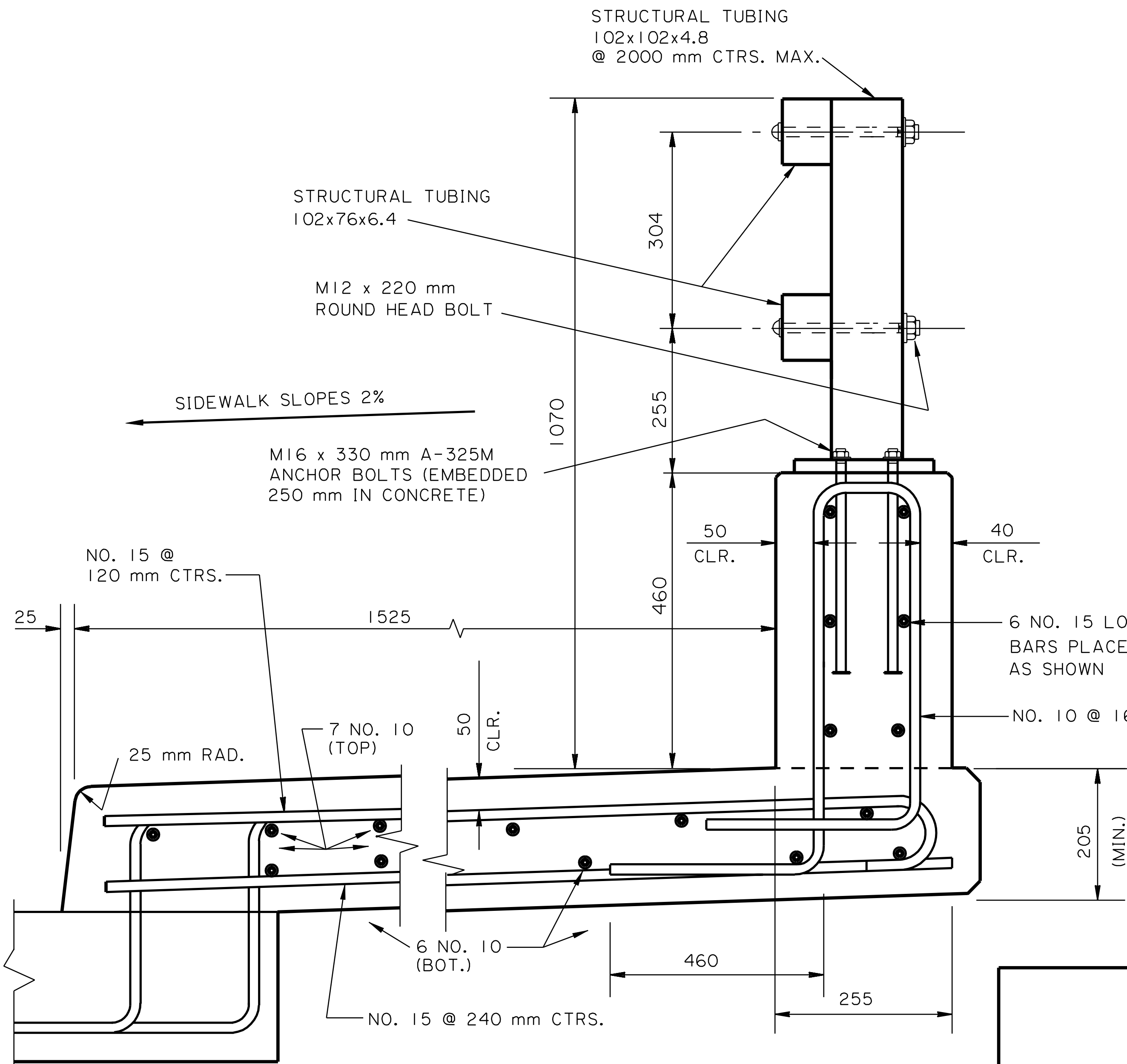
2 OF 2



NOTES:

1) CONCRETE TO BE CLASS AA
2) REINFORCING STEEL TO BE GRADE 420

NEW JERSEY SHAPE
(PL-2)

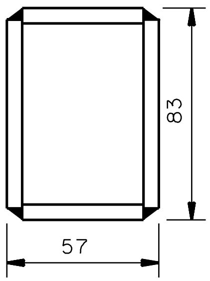


NOTES:

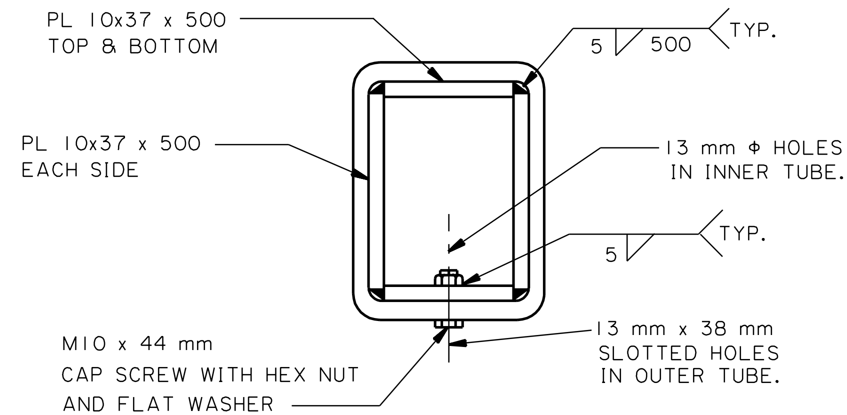
1) CONCRETE TO BE CLASS AA
2) REINFORCING STEEL TO BE GRADE 420 AND TO MATCH CORROSION PROTECTION OF DECK STEEL.
3) ALL STRUCTURAL TUBING IS ASTM A-500M, GRADE B MATERIAL.
4) STRUCTURAL STEEL TO BE A-709M.
5) ALL STRUCTURAL STEEL AND HARDWARE SHALL BE HOT-DIPPED GALVANIZED AFTER FABRICATION.
6) THE LENGTH OF THE RAIL SEGMENT SHALL NOT EXCEED THIRTY METERS.
7) RAIL SPLICES ARE TYPICALLY AT 6000 mm WITH CENTER OF SPLICES 500 mm FROM NEAREST POST. NO SPLICES ARE ALLOWED WITHIN 5000 mm FROM OPEN JOINT.
8) JOINTS ARE REQUIRED AT 30 000 mm CENTERS (MAX.)

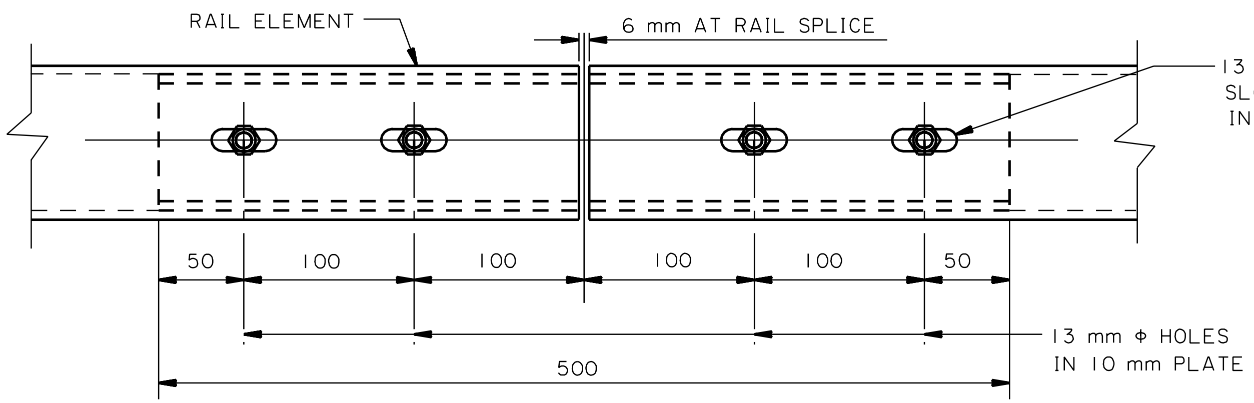
RAILING / SIDEWALK
(PL-1)

1 OF 3



FINISHED DIMENSIONS
OF INNER SLEEVE

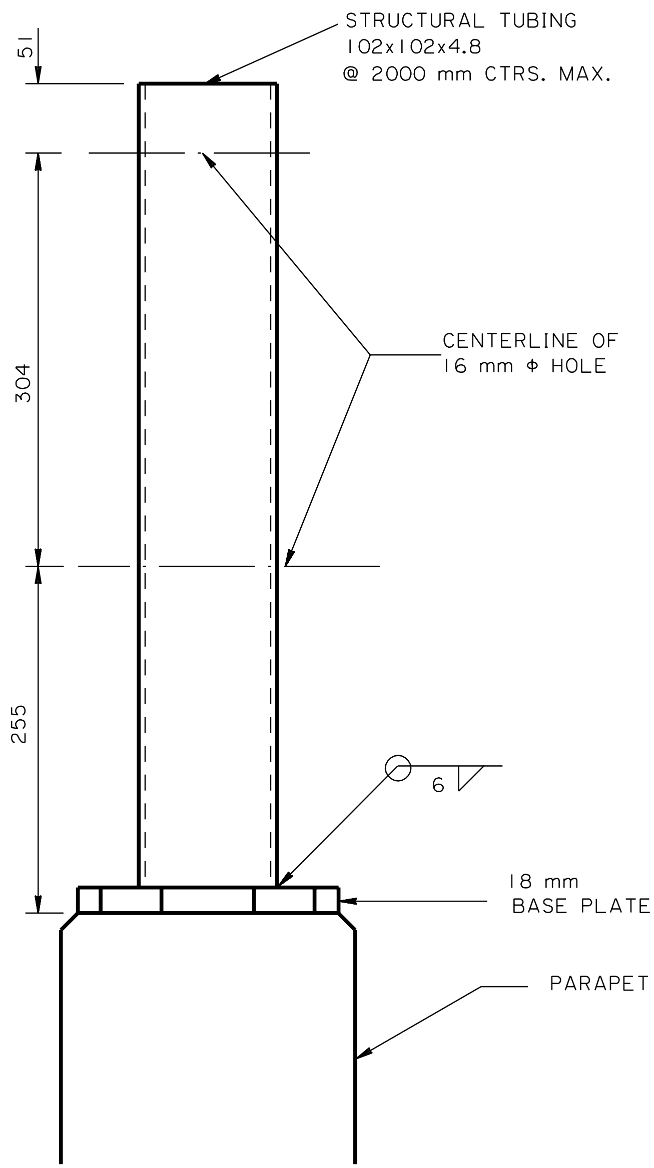


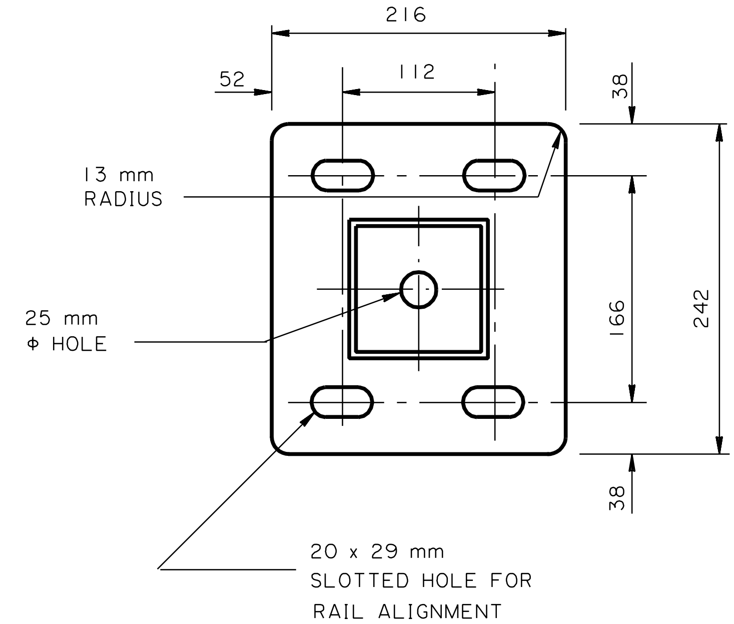


BOTTOM PLATE INTERNAL SPLICE

RAILING / SIDEWALK
(PL-1)

2 OF 3





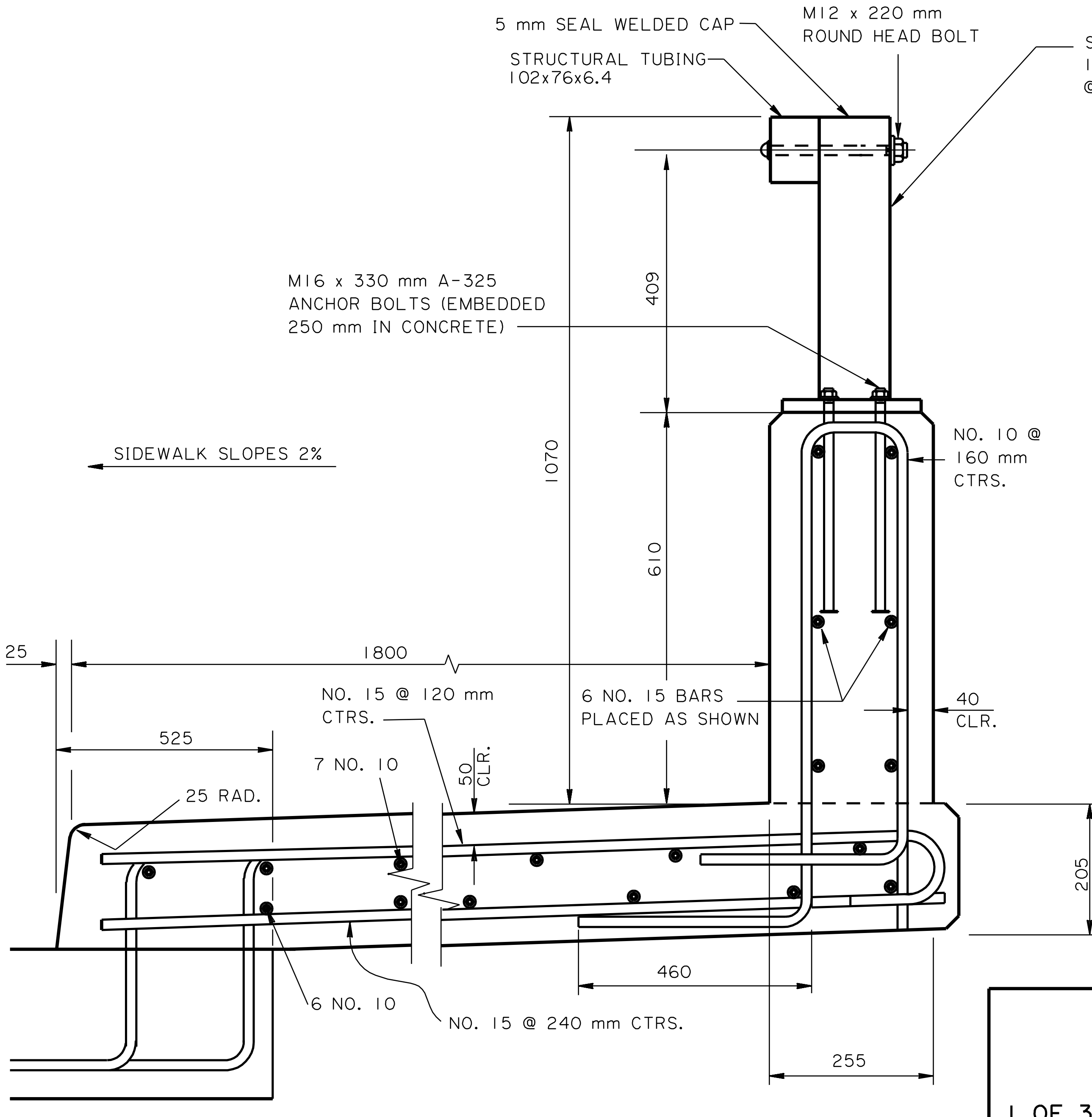
BASE PLATE DETAIL

NOTE:

INTERMEDIATE RAIL SPLICES ARE AT EVERY 6000 mm WITH CENTER OF CONNECTION AT SPAN QUATER POINTS. RAIL SPLICES SHALL NOT BE ALLOWED ON THE LAST TWO SPANS AT THE END OF THE SEGMENTS.

RAILING / SIDEWALK
(PL-1)

3 OF 3



NOTES:

1) CONCRETE TO BE CLASS AA

2) REINFORCING STEEL TO BE GRADE 420 AND TO MATCH CORROSION PROTECTION OF DECK STEEL.

3) ALL STRUCTURAL TUBING IS ASTM A-500M GRADE B MATERIAL.

4) STRUCTURAL STEEL TO BE A-709M.

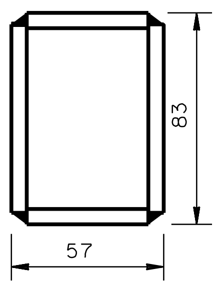
5) ALL STRUICTURAL STEEL AND HARDWARE SHALL BE HOT DIPPED GALVANIZED AFTER FABRICATION.

6) JOINTS ARE REQUIRED AT 30 000 mm

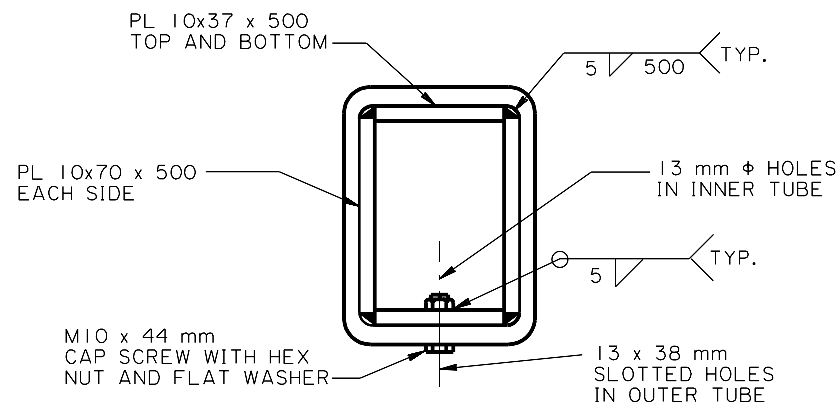
7) RAIL SPLICES ARE TYPICALLY AT 6000 mm WITH CENTER OF SPLICE 500 mm FROM NEAREST POST. NO SPLICES ARE ALLOWED WITHIN 5000 mm FROM OPEN JOINT.

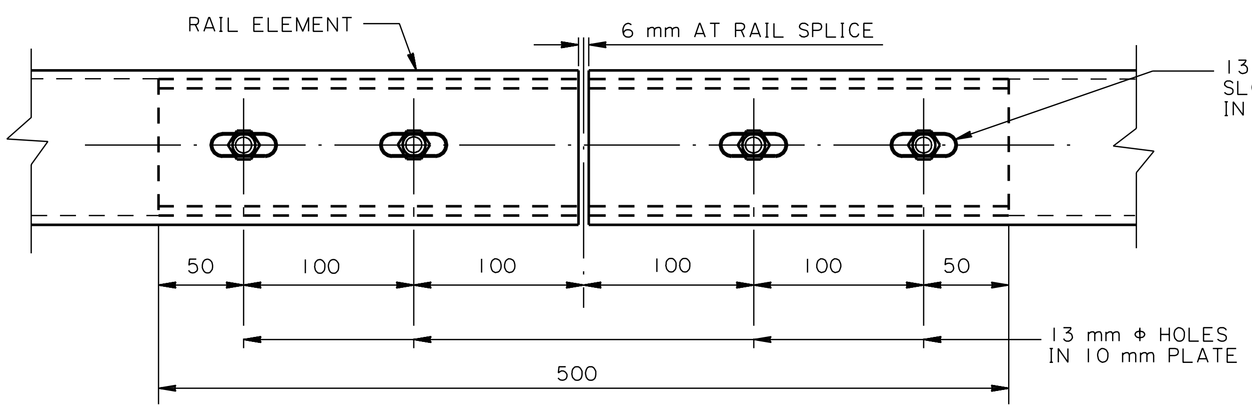
8) IF CONDITIONS EXIST WHERE 1825 mm WIDTH CAN NOT BE PROVIDED, A MINIMUM WIDTH OF 1550 mm CAN BE USED.

RAILING / SIDEWALK (PL-2)



FINISHED DIMENSIONS OF INNER SLEEVE

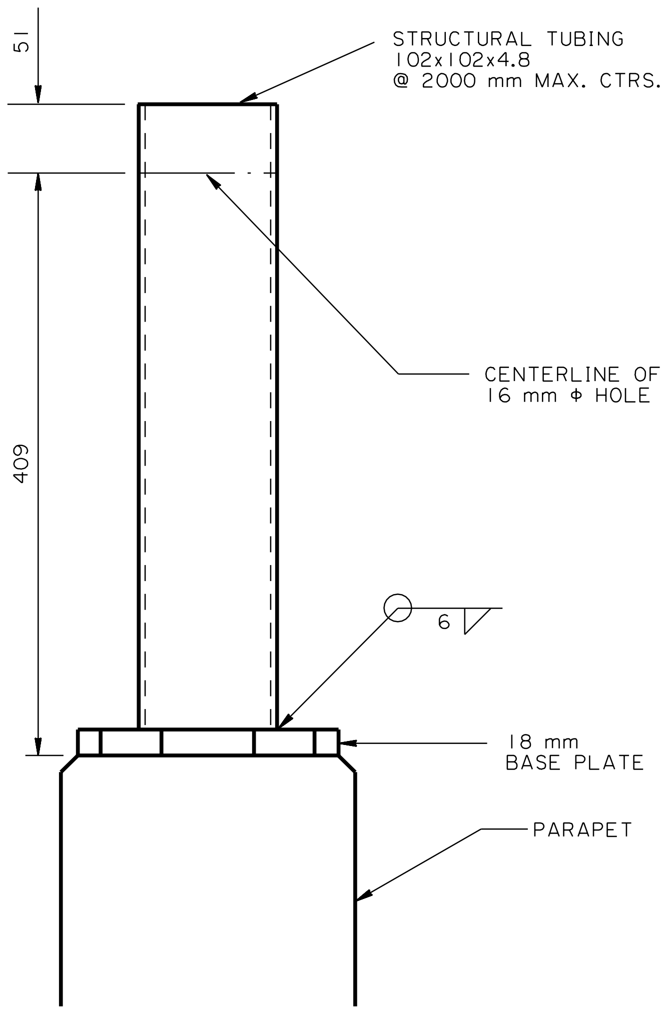


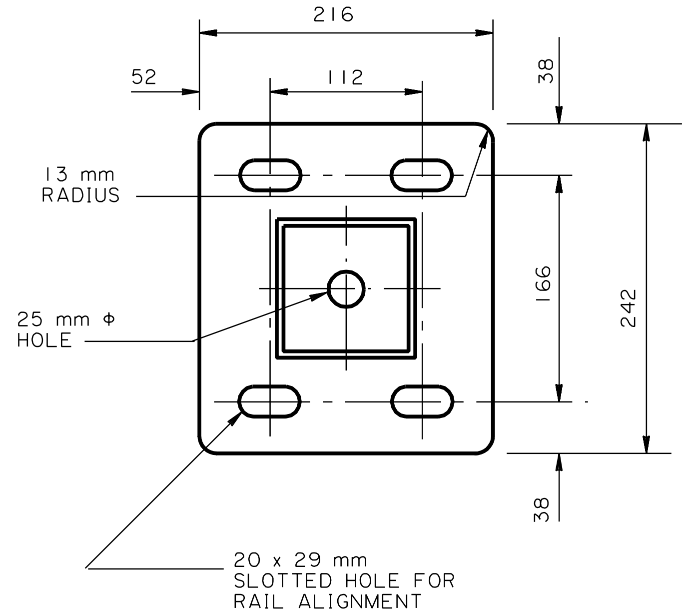


BOTTOM PLATE INTERNAL SPLICE

RAILING / SIDEWALK
(PL-2)

2 OF 3





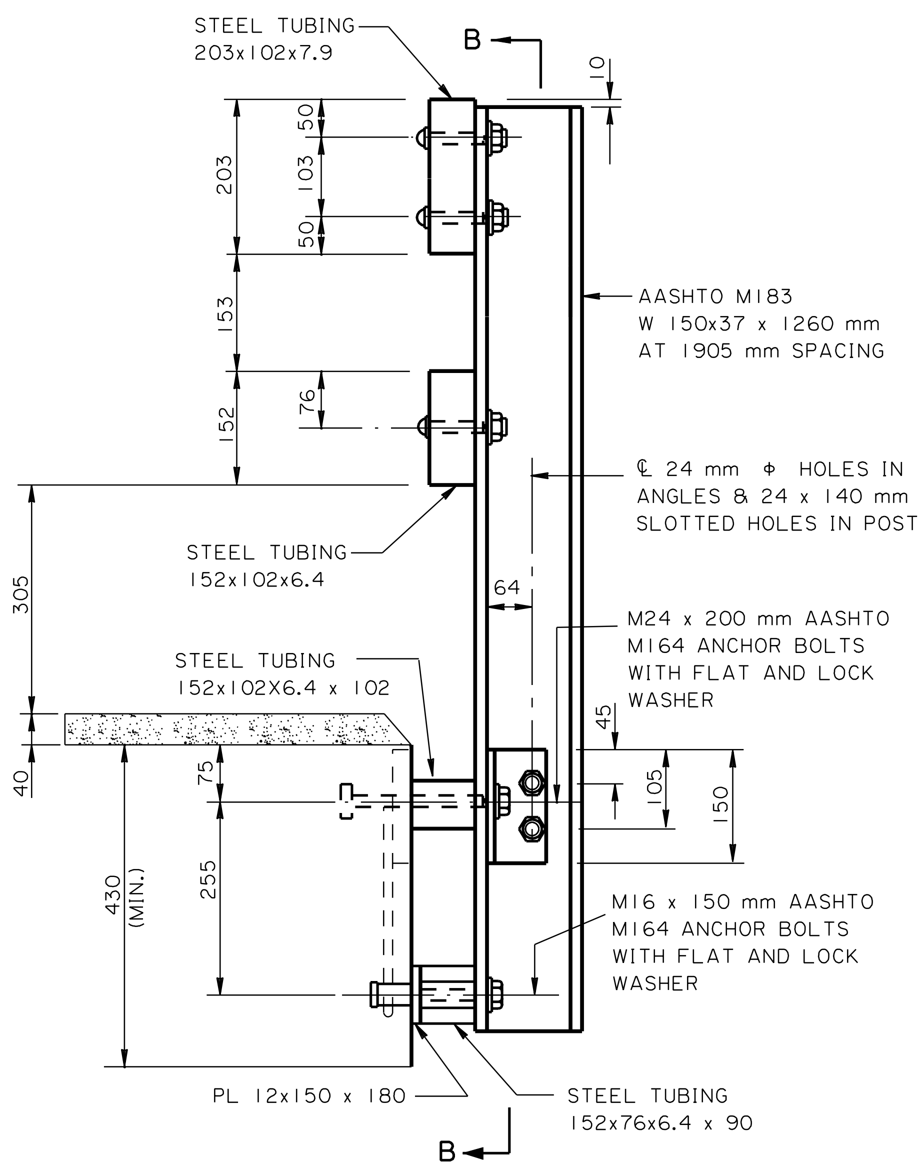
BASE PLATE DETAIL

NOTE:

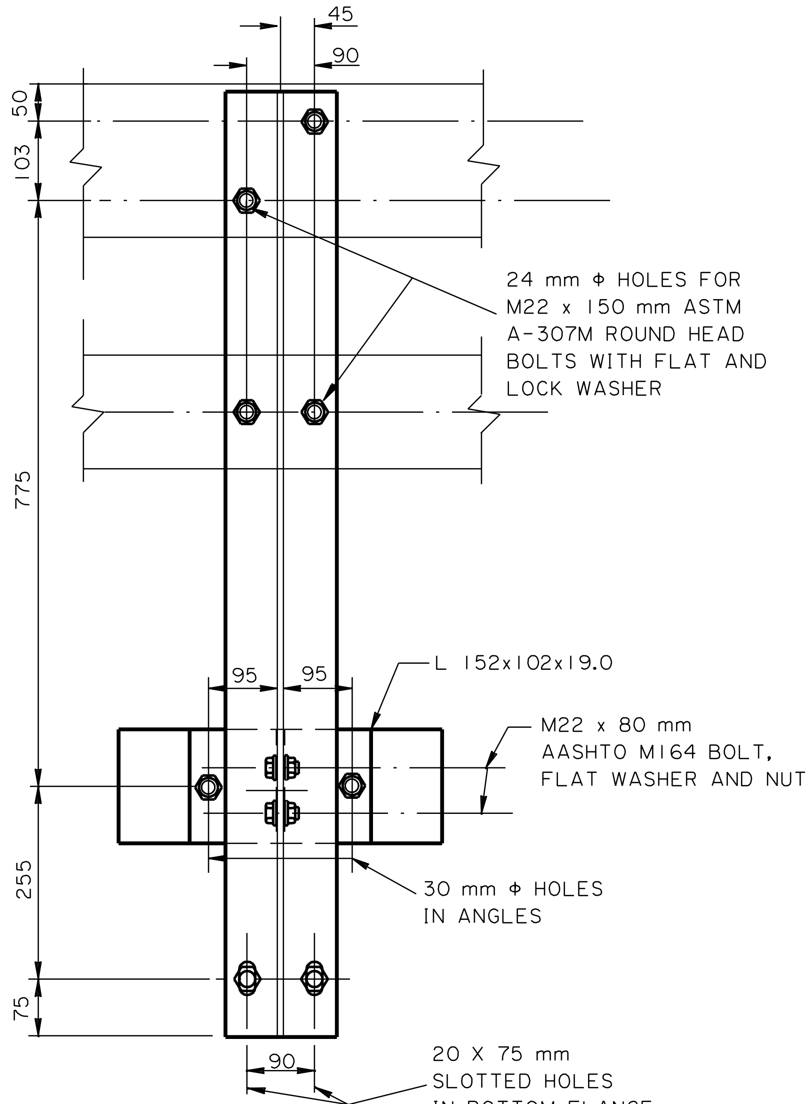
INTERMEDIATE RAIL SPLICES ARE AT EVERY 6000 mm WITH CENTER OF CONNECTION AT SPAN QUARTER POINTS. RAIL SPLICES SHALL NOT BE ALLOWED ON THE LAST TWO SPANS AT THE END OF THE SEGMENTS.

RAILING / SIDEWALK (PL-2)

3 OF 3



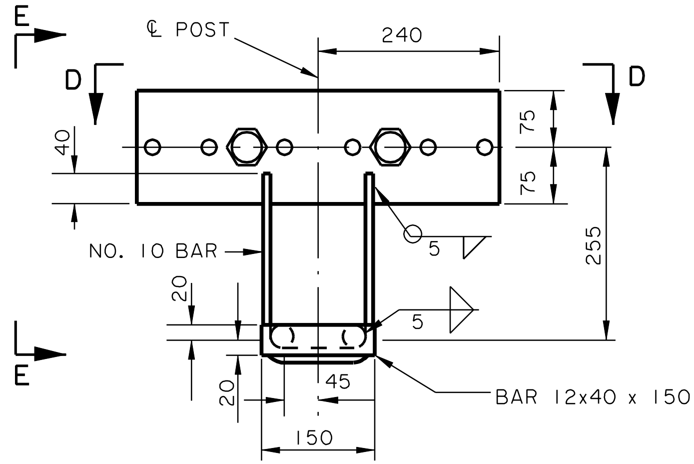
SECTION AT RAIL POST

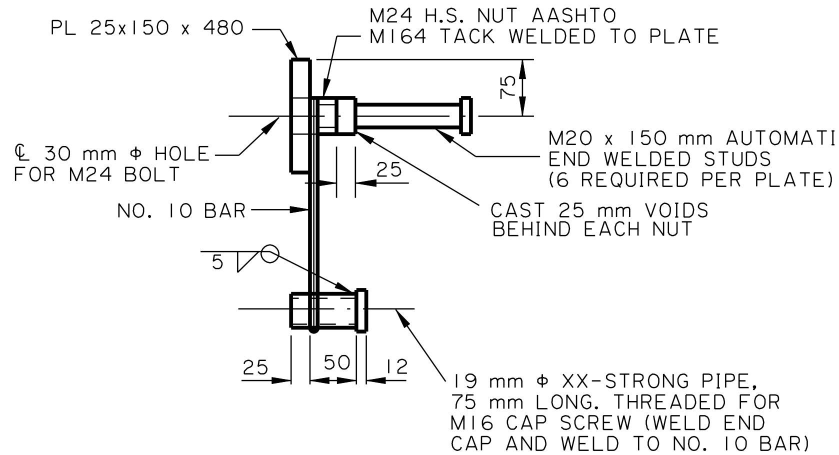


SECTION B-B

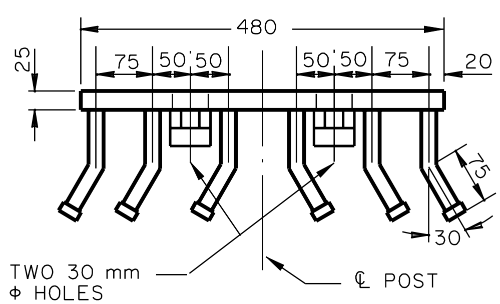
SIDE MOUNTED RAIL
(PL-2)

1 OF 3





VIEW E-E



VIEW D-D

NOTES:

1) ALL STRUCTURAL TUBING IS ASTM A-500M, GRADE B.

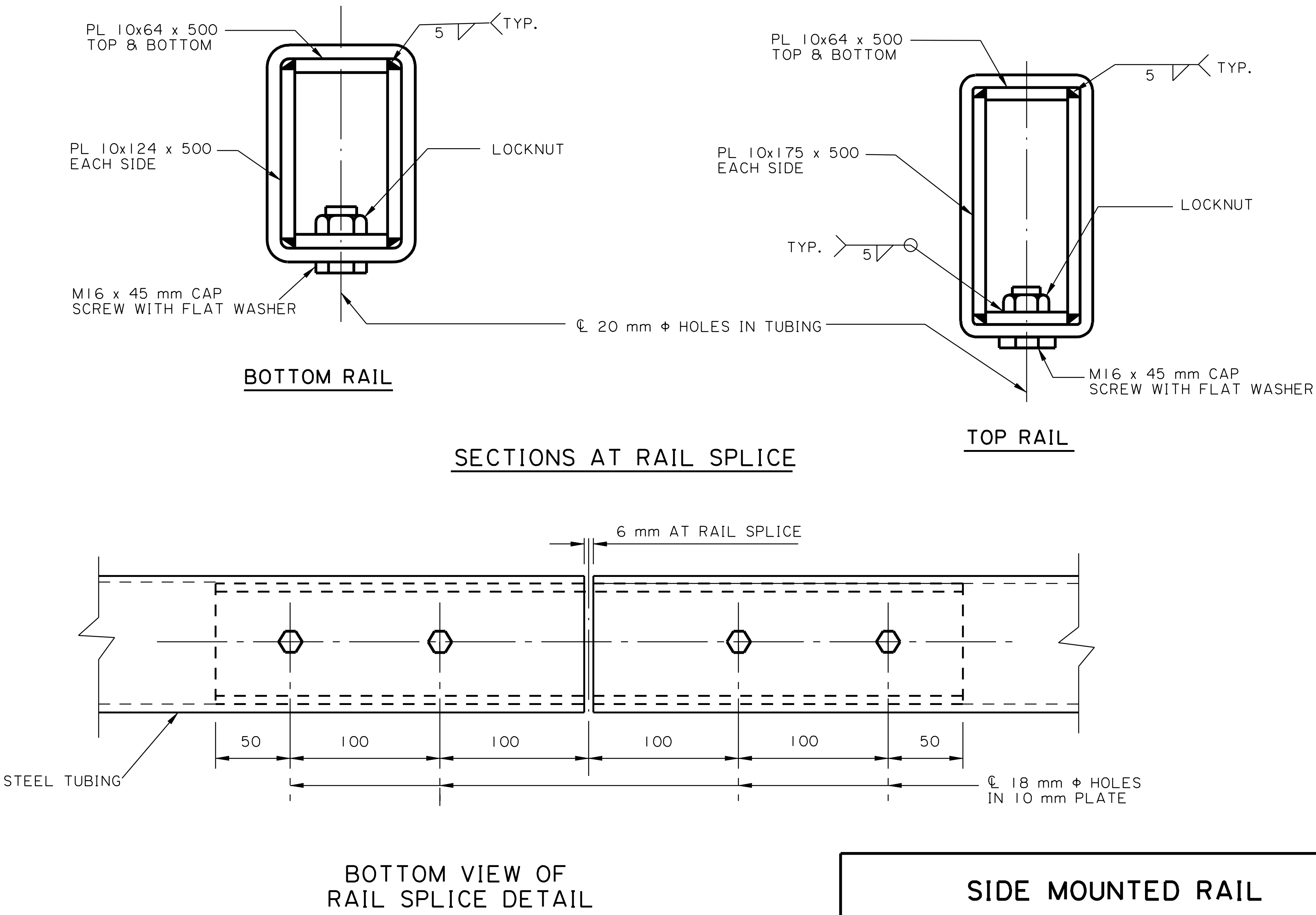
2) STRUCTURAL STEEL TO BE GRADE A-709M

3) ALL STRUCTURAL STEEL AND HARDWARE SHALL BE HOT DIPPED GALVANIZED AFTER FABRICATION.

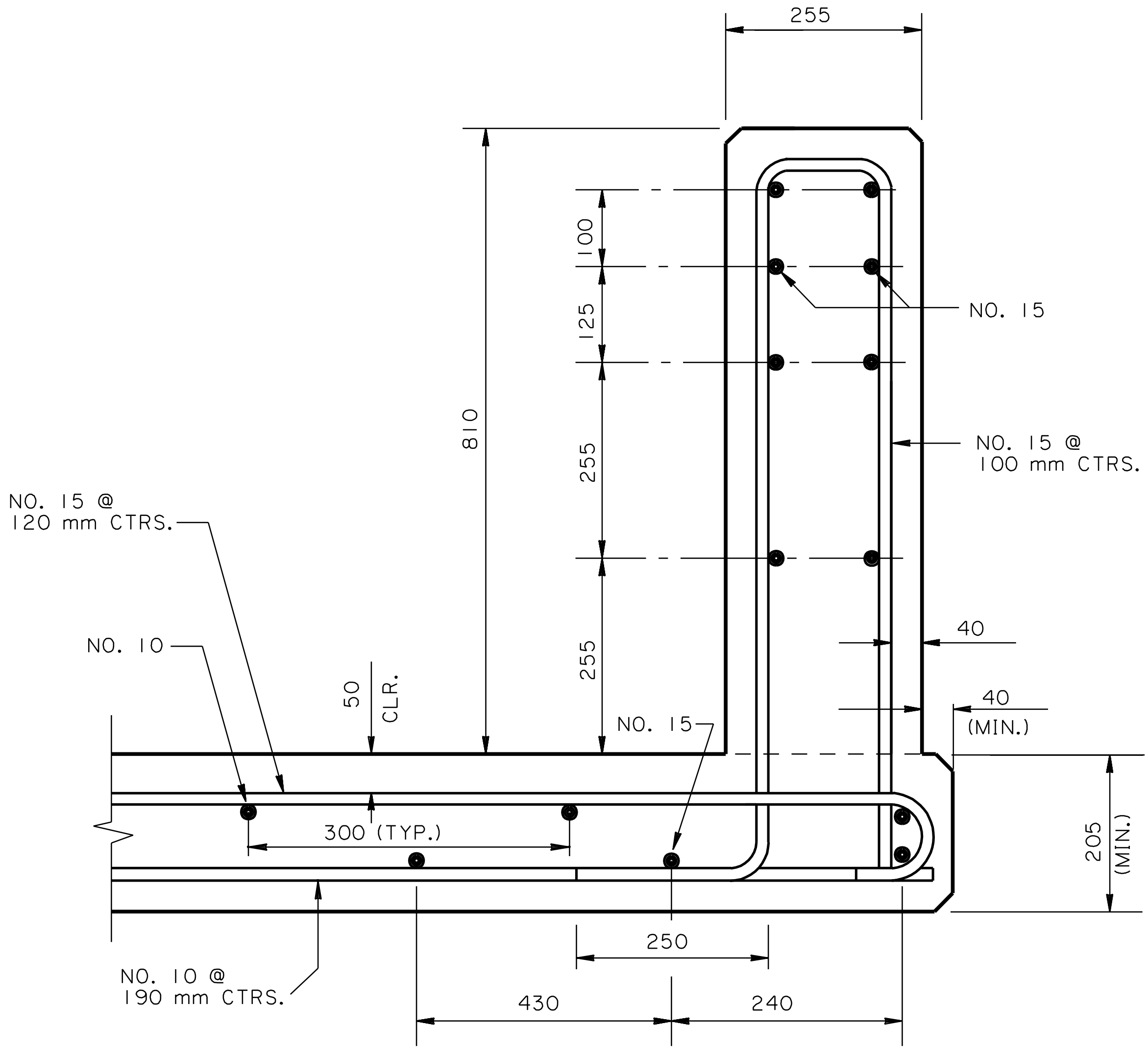
ANCHOR DEVICE

SIDE MOUNTED RAIL
(PL-2)

2 OF 3



SIDE MOUNTED RAIL
(PL - 2)

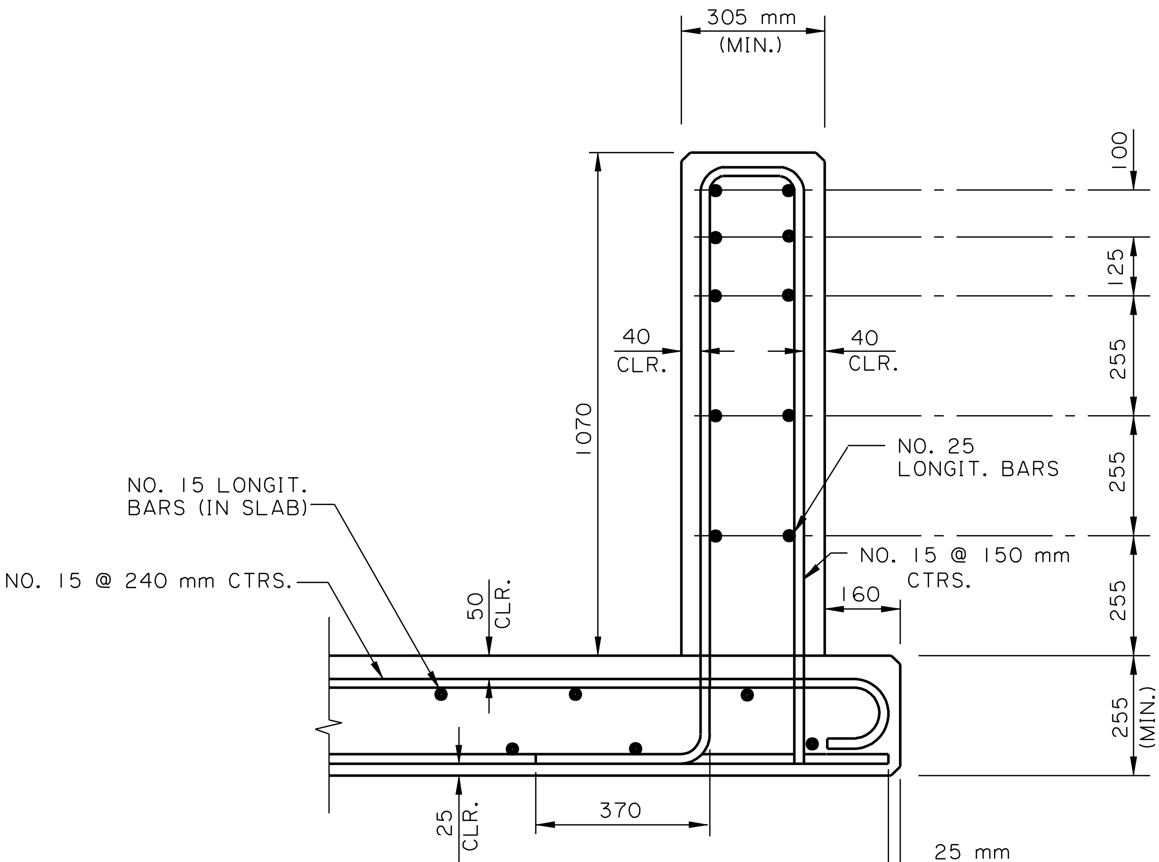


NOTES:

1) CONCRETE TO BE CLASS AA

2) REINFORCING STEEL TO BE GRADE 420

VERTICAL WALL
(PL-2)



NOTES:

1) CONCRETE TO BE CLASS AA

2) REINFORCING STEEL TO BE GRADE 420

VERTICAL WALL
(PL-3)

TEMPORARY CONCRETE BARRIERS

Purpose and Scope

The following sets forth the DOTD's policy with regard to the installation of temporary concrete barriers on highway construction projects. The policy is applicable to all state and federal projects involving the temporary operation of two-way traffic on one side of a multi-lane highway or bridge.

Policy

Temporary concrete barriers generally shall be required to separate opposing traffic flow on interstate construction projects where shoulders exist on both sides of the lanes on which detour traffic will be directed. Length of detour will be limited to approximately eight (8) kilometers or less. Temporary barriers are also used to provide a positive barrier between workers and traffic.

Temporary barriers in other situations will be considered on a case by case basis and may be used upon written approval of the Chief Engineer.

Temporary barriers normally will not be used on non-control access highways. Barrier placement on multi-lane bridges shall be decided on a project by project basis after evaluation of suitable construction alternatives.

If temporary barriers are used for a bridge application, the Engineer should consider a more rigid anchorage of the temporary barrier to the bridge deck when deemed necessary.

BRIDGE DECKS

There are currently three types of bridge riding surfaces being used by the Department:

1. Full Depth Cast-in-Place (slab span[1] or girder deck[2])

2. Stay-in-Place Forms with cast-in-place concrete topping

   a) Slabs - always utilize concrete panels

   b) Decks

      1) Precast Concrete Panels

      2) Stay-in-Place Steel Forms

      3) Grid Floor

The following standard plans or standard details relating to bridge decks should be included in the bridge plans where applicable:

1. Slab Spans (various roadway widths and skew angles)

2. Miscellaneous Deck and Girder Details

3. Optional Deck Details

4. Grid Floor

5. Strip Seal Joint Details

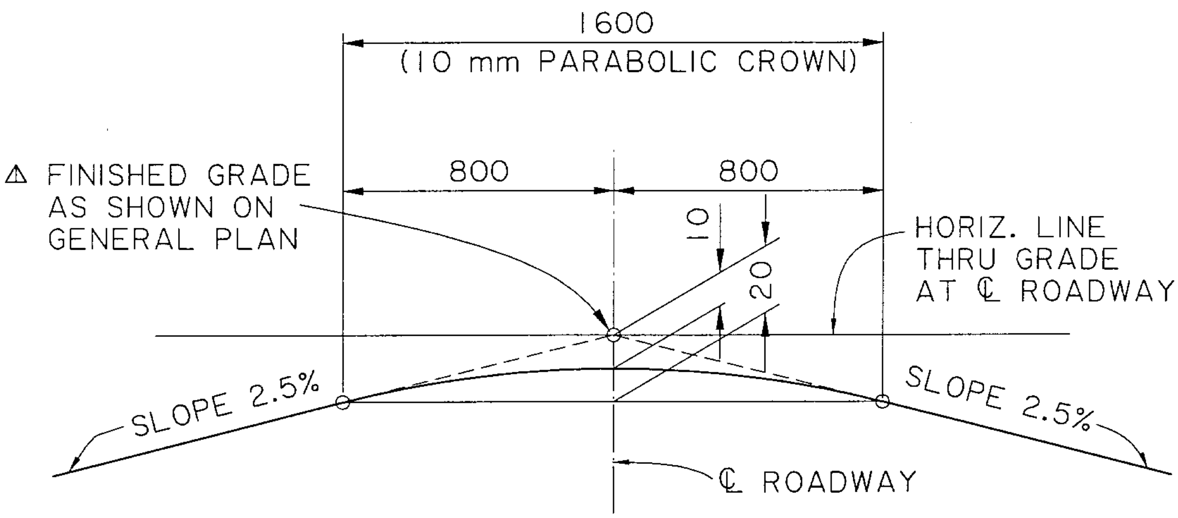
[1] Slab defined as structural riding surface between substructure members.
[2] Deck defined as structural riding surface between superstructure members.

ROADWAY CROWNS

1. One-way traffic bridges shall have a single tangent slope of **2.5%**.

2. Two-way traffic bridges shall have two-way tangent slopes of **2.5%** connected by a **1600 mm** parabolic crown section, except for projects utilizing slab span metric standards which presently show a soft conversion of the crown as **1220 mm**.

3. Bridge deck crowns shall match connecting roadway crowns parabolic crown except for special cases.



SKETCH SHOWING PARABOLIC CROWN

△ UNLESS OTHERWISE NOTED IN PLANS

SUPERELEVATION

Reasons for Superelevation of Curves

Centrifugal force causes a vehicle moving in a circular path to slide away from the center of the curve. This tendency can be reduced to acceptable levels by increasing the cross slope of the roadway surface away from the curve center sufficiently so that the vehicle weight component parallel to the roadway surface approximates the centrifugal force component parallel to the roadway surface.

The design of superelevation for highway curves will be in accordance with the latest edition of *A Policy on Geometric Design of Highways and Streets*, AASHTO, Washington D.C.

Definition of Terms

ACR is the abbreviation for adverse crown removed. It is where the outside travel lane is rotated to a 0.00% cross slope and the inside travel lane remains at the normal crown cross slope.

$\mathbf{e_{max}}$ is the symbol for the maximum superelevation rate of the roadway cross section. Louisiana normally limits this value to10.0% for rural and 4.0% for urban roadway.

e is the symbol for rate of superelevation for the roadway cross slope and is a function of e_{max}, design speed, and radius of curve. Values are generally selected from Tables III-7 to 11 in AASHTO.

MRG is the acronym for maximum relative gradient between the profile edge of two-lane travel way and the centerline. Its value is a function of design speed. The fractional value, given as a ratio, is **MRS**, or Maximum Relative Slope. MRS values are generally selected from Table III-13 in AASHTO.

NC is the abbreviation for normal crown section. In tables it designates curves with radii long enough that elimination of adverse cross slope or use of superelevation is not necessary. AASHTO tables are based on 1.50% cross slope. The designer should be aware that larger curve radii than that shown with NC designation may be required when using cross slopes of 2.50%.

R stands for the radius of a horizontal curve.

RC is the abbreviation for remove crown cross section. In tables it designates curves with radii long enough that removal of adverse cross slope and superelevating the entire roadway at normal crown inside slope is adequate. AASHTO tables are based on 1.50% cross slope. The designer should be aware that smaller curve radii than that with a RC designation may be satisfactory when using a cross slope of 2.50%.

Superelevation runoff is the general term denoting the length of highway needed to accomplish the change in cross slope from a section with adverse crown cross slope removed to a fully superelevated section, or vice versa.

Tangent runout is the general term denoting the length of highway needed to accomplish the change in cross slope from a normal crowned cross section to a section with the adverse crown cross slope removed, or vice versa.

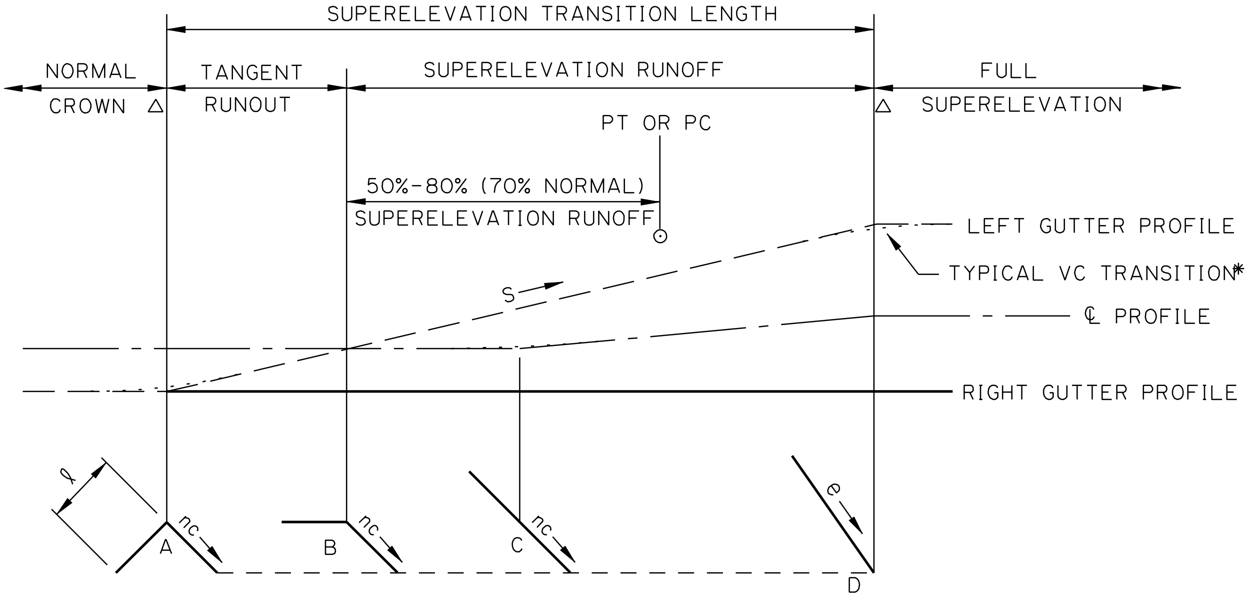
V is the assumed design speed.

Superelevation Rotation: In order to transition from a normal crown section to a fully superelevated section, a superelevation transition detail must be provided. The two primary methods used in LADOTD Bridge Design for attaining superelevation for curves are: 1) traveled way revolved about the centerline, and 2) traveled way revolved about the low gutter [in bridges with superelevation, it is common practice to profile the gutterline rather than the travel lane profile].

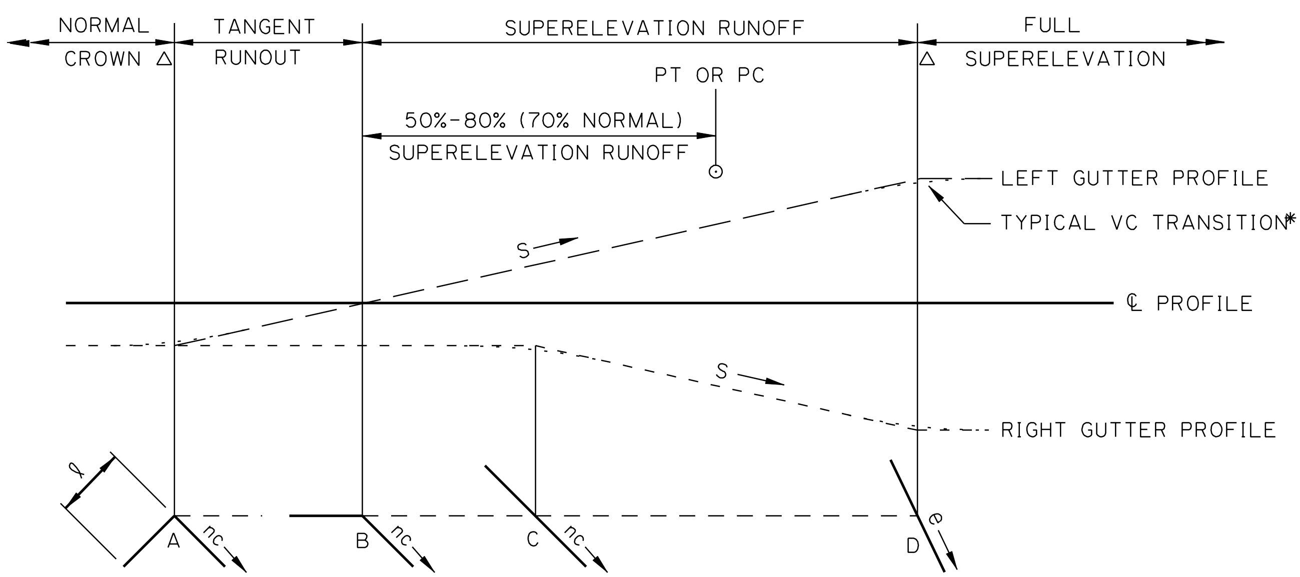
The method of rotating the traveled way about the centerline is widely used in design because the required change in elevation of the gutterlines is made with less distortion and it produces a balance with right-of-way taking. However, with one-half of the required elevation change made at each gutterline, vertical clearance reduction, drainage problems (if low gutter drops below natural ground), and driver apprehension of a dip in the roadway due to lowering of the low gutter of the roadway must be considered.

The method of rotating the traveled way about the low gutter is frequently used and the required change in cross section is accomplished by raising the outside gutter profile. This method is preferable when the low gutter line profile is a major control, as for drainage or vertical clearance above waterways. This method produces the greatest distortion of the high gutterline profile and could result in additional right of way taking due to the increase in fill height associated with the increase in elevation of the high gutter line.

Regardless of which method is used to superelevate the roadway surface for horizontal curves on projects, close coordination should be maintained between the Road and Bridge Design Sections, since the superelevation transitions usually affect both bridge and roadway designs.



PROFILE USING LOW GUTTER ROTATION



PROFILE USING CENTERLINE ROTATION

* VC LENGTH (METERS) = $\frac{\text{DESIGN VELOCITY (Km/h)}}{5}$

nc = NORMAL CROWN
e= RATE OF ROADWAY SUPERELEVATION
S= MAX. RELATIVE SLOPE BETWEEN THE EDGE OF ROADWAY PROFILE AND THE REFERENCE PROFILE.
△= OUTSIDE EDGE BEGINS TRANSITION VC/2 BEYOND THIS POINT

Rev. 10/15/99

SUPERELEVATION DEVELOPMENT OPTIONS

Design Considerations

1. As a general rule, 60% to 80% of the superelevation runoff should be achieved on the tangent prior to the PC or after the PT of the curve (70% is normal, 50% is allowed for back to back curves).

2. For traveled ways wider than two lanes, adjustments should be made for lane widths as per AASHTO. In Bridge Design, the width of shoulders is included in the overall traveled way widths for adjustment purposes.

3. Angular breaks in the profile control lines should be rounded in final design by insertion of short vertical curves with minimum lengths (meters) approximately equal to V/5, with the design speed in kilometers per hour. These short vertical curves should be located so that their PVC and / or PVT are located at bent centerlines, when practical. This may require an elongation of the superelevation diagram.

4. A review by graphical plots of edgelines and centerline should be made to ensure smooth lines and to identify any flat areas. Kinks can be avoided by lengthening curves. Inadequate roadway slope creates potential drainage hazards (one such area is the point of adverse crown removed on a 0.00% grade bridge) that can be reduced by minimizing the superelevation transition lengths or providing additional drains in the area.

5. Bridge length, profile grade line, and superelevation rotation should all be referenced to a common baseline when possible.

6. All plan sheets should be detailed consistent with the examples shown in this manual when possible. It should be noted that the superelevation examples on the following pages are given as a guide; they are but one interpretation of AASHTO. For all projects, but especially the more complex, the engineer is encouraged to apply information contained in *A Policy on Geometric Design of Highways and Streets*, AASHTO, Washington D.C. to develop suitable superelevation transition diagrams.

Lane Factor (adjustment for pavement width ①)	
Length (L) (distance from rotation line to edge of rotated surface) (m)	**LF**
3.6	**1.0**
4.5	**1.1 ②**
5.4	**1.2**
6.0	**1.3 ②**
6.6	**1.4 ②**
7.2	**1.5**
10.8	**2.0**
> 10.8	**extrapolate**

①From page 180, A Policy on Geometric Design of Highways and Streets, 1994 AASHTO.

②Interpolated value

Maximum Relative Slope (between longitudinal edges of 3.6 m lane ③)	
Design Speed **(V) km/h**	**MRS**
30	**1:133**
40	**1:143**
50	**1:150**
60	**1:167**
70	**1:182**
80	**1:200**
90	**1:210**
100	**1:222**
110	**1:238**
120	**1:250**

③ Table III-13, A Policy on Geometric Design of Highways and Streets, 1994 AASHTO

Examples

Given: V = 100 km /h
R = 600 m
nc = 0.025 (standard cross-slope)
e_{max} = 10% (DOTD rural standard)
e = . 078 (from AASHTO Table III-10)
MRS = 1 / 222 (from AASHTO Table III-13)
≤ rise / minimum run (Value based on 2 second min. runoff length,p.177.
≤ (lane width*e)/minimum table runoff Obtain 2-lane minimum runoff value from
≤ (3.6 * 0.078) / 62 = 1:221 Table III-10 and incorporate if geometry permits.)

Rotate about centerline

A) two 3.6 m lanes
no shoulders
L = 3.6 m (distance from rotation line to edge of rotated surface)
LF = 1.0 (function of L and page 180, AASHTO)

run = rise / slope
AB = [lane * nc / MRS] * LF = 3.6(0.025) 222 (1.0) = **0.09**(222) ≅ 20 m (runout)
BD = [lane * e / MRS] * LF = 3.6(0.078) 222 (1.0) = **0.28**(222) ≅ 62 m (matches Table III-10)
AD = superelevation transition length **82 m**
g = rise / run = (nc + e) * L / AD = (.025 + .078)3.6 / 82 = **0.004522 m/m**

Remainder of the superelevation diagram geometry can be determined from the **control values**.

B) four 3.6 m lanes
no shoulders

L = 3.6 * 2 = 7.2 m (distance from rotation line to edge of rotated surface)
LF = 1.5 (function of L and page 180, AASHTO)

AB = 3.6 (0.025) 222 (1.5) ≅ 30 m
BD = 3.6 (0.078) 222 (1.5) <u>≅ 94 m</u> (matches Table III-10)
AD = 124 m
g = (0.025 + 0.078)7.2 / 124 = 0.005981 m/m

C) two 3.6 m lanes
two 3.0 m shoulders

L = 3.6 + 3.0 = 6.6 m
LF = 1.4

AB = 3.6 (0.025) 222 (1.4) ≅ 28 m
BD = 3.6 (0.078) 222 (1.4) <u>≅ 88 m</u>
AD = 116 m
g = (0.025 + 0.078)6.6 / 116 = 0.005860 m/m

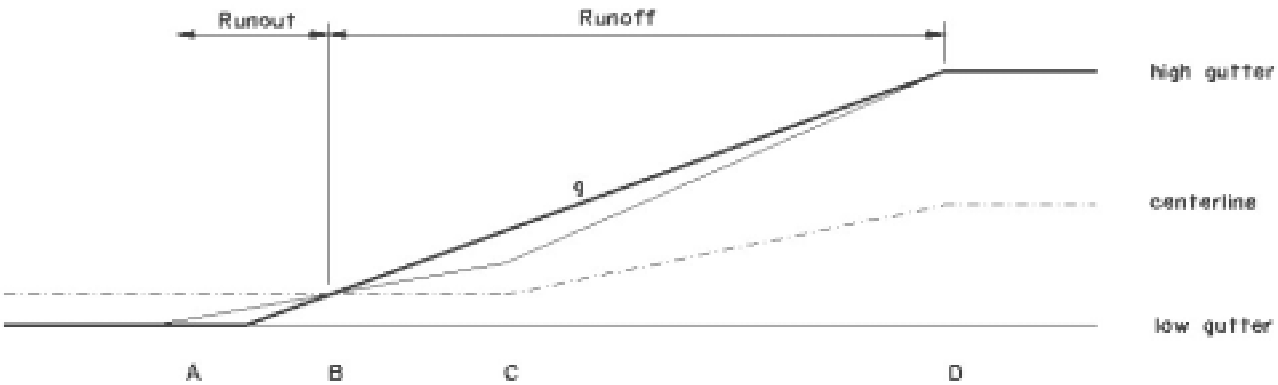
<u>Rotate about low gutter</u>

[Page 187 of *A Policy on Geometric Design of Highways and Streets*, 1994 AASHTO, indicates that when rotation is about roadway edge, runoff lengths are similar to those for a centerline rotation with the same roadway width. This is consistent with page 177 where the MRG between profiles of edges of two-lane traveled ways is allowed to double. One way to account for this is by using a lane factor (LF) based on an adjusted length (L' = ½ L) when calculating the required runoff length.]

D) two 3.6 m lanes
no shoulders

L = lanes + shoulders = 2 * 3.6 = 7.2 m (dist. from rotation line to edge of rotated surface)
L' = ½*L = ½* 7.2 = 3.6 m (corrected width for determining LF for low gutter rotation)
LF = 1.0 (function of L' and page 180, AASHTO)
BC = [lane * nc / MRS] * LF = 3.6 (0.025) 222 (1.0) = **0.09**(222) ≅ 20 m
CD = [lane * (e-nc) / MRS] * LF = 3.6 (0.078-0.025) 222 (1.0) <u>≅ 42 m</u>
BD = [lane*e / MRS]*LF = 3.6 (0.078) 222 (1.0) = **62 m** (matches Table III-10)
g = [(runout width)*nc + L(e-nc)] / BD
= [(3.6*0.025) + 7.2*(0.078-0.025)] / 62 = (.09 + .3816) / 62 = **.4716** / 62 = **.007606 m/m**

[Note that runoff length (BD) is independent of initial crown cross slope (we would obtained 62 m in the above example if nc had been 0.000 or 0.015 in lieu of 0.025). Also note the “g” calculated is the gradient associated with the allowed runoff from Table III-10 but is dependent on initial crown. When an initial crown exists, the preliminary gradient of BC is one half the value of the preliminary gradient for CD (this is due to the shift in the reference rotation line from centerline to low gutterbeyond point C, and the corresponding rise of the initial reference line beyond point C).



Thus “g” (as calculated above) is an average value. Page 184 of AASHTO states that a uniform edge slope is desirable. When the desired relative slopes are not possible, the runout length should be at least equal to those required for a curve with maximum superelevation where the same relative slope for the tangent runoff and runout are retained. With this as a guideline, we can conservatively use the average “g” as calculated above for both runout and runoff. This insures that in all occurrences, a single gradient is used, that the runout is longer than policy minimum, and “g” is less than the maximum.]

In our case:

runoff = [e*lane / MRS]*LF
= [0.078*3.6*222] * 1.0 = 62 m

g_{max} = (e*L) / runoff
= 0.078*7.2 / 62 = 0.00906 > g (=0.007606 m/m)

If we use a flatter gradient than g_{max} to determine runout length, our runout length should always be longer than minimum required. To confirm this assumption, calculate and compare the two lengths one time.

$runout_{min}$ = (nc*runout width)/g_{max}
= 0.025*3.6 / 0.00906 = 9.936 m

AB = (nc*runout width) / g
= (.025 *3.6) / 0.007606 = **11.833 m** > $runout_{min}$

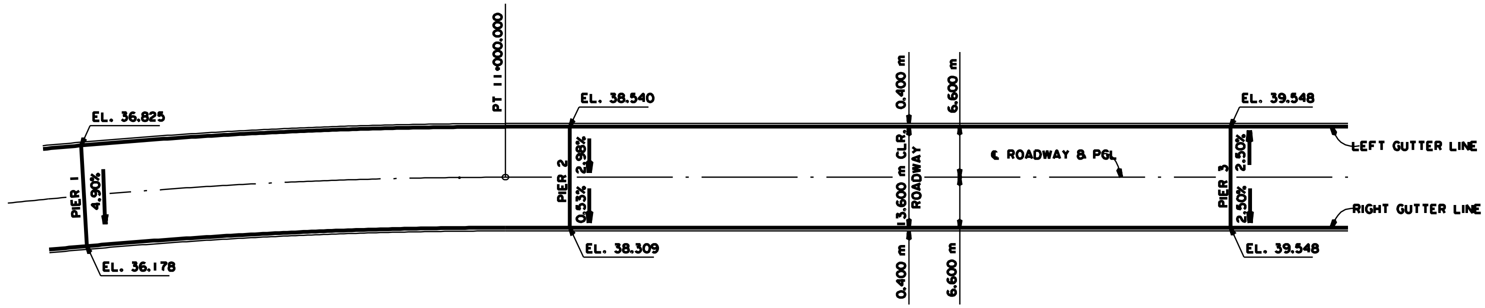
The remainder of the superelevation diagram geometry can be determined from the **control values**.

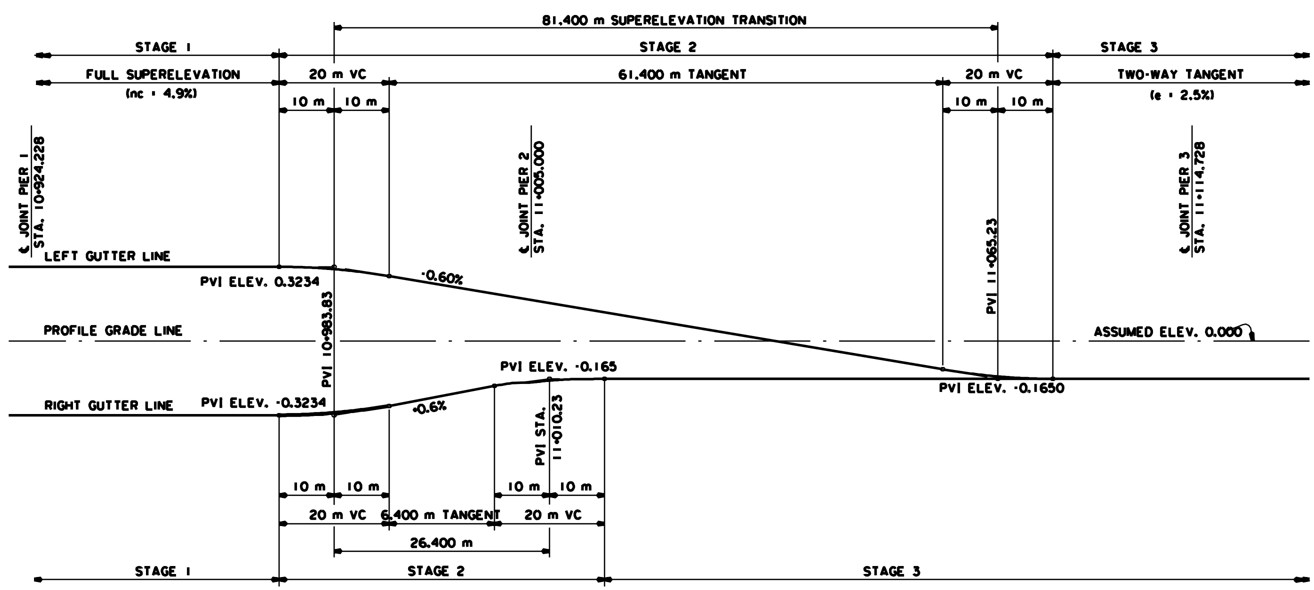
With our assumptions confirmed, we can reduce the steps necessary to find control values in subsequent problems.

E) two 3.6 m lanes
two 3.0 m shoulders

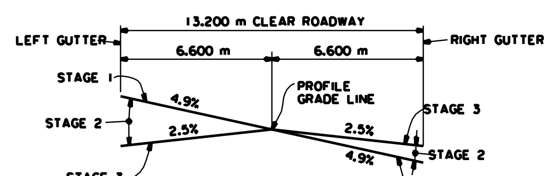
L = lanes + shoulders = 2 * (3.6 + 3.0) = 13.2 m
L' = ½*L = ½* 13.2 = 6.6 m (corrected width for determining LF for low gutter rotation)
LF = 1.4 (function of L' and page 180, AASHTO)

BD = [e*lane / MRS]*LF = 0.078(3.6) 222(1.4) = **88 m**
g = [(runout width)*nc + L(e-nc)] / BD
= [(3.6 + 3.0)*0.025 + 13.2*(0.078-0.025)] / 88
= (**0.165** + .6996) / 88 = **0.8646** / 88 = **0.009825 m/m**
$\leq$ e*L/BD = (0.078*13.2) /88 = .0117 okay
AB = BC = [(nc* runout width) / g] = [0.025*(3.6 + 3.0) /0.009825] = 0.165 /0.009825
= **16.79 m**





CORRECTION TABLE				
LOCATION	℄ JOINT STATION	PROFILE GRADE ELEV.	CORRECTION LEFT GUTTER	CORRECTION RIGHT GUTTER
PIER 1	10+924.228	36.5015	+0.3234	-0.3234
PIER 2	11+005.000	38.3439	+0.1964	-0.0348
PIER 3	11+114.728	39.7132	-0.1650	-0.1650



TRANSITION SCHEMATIC

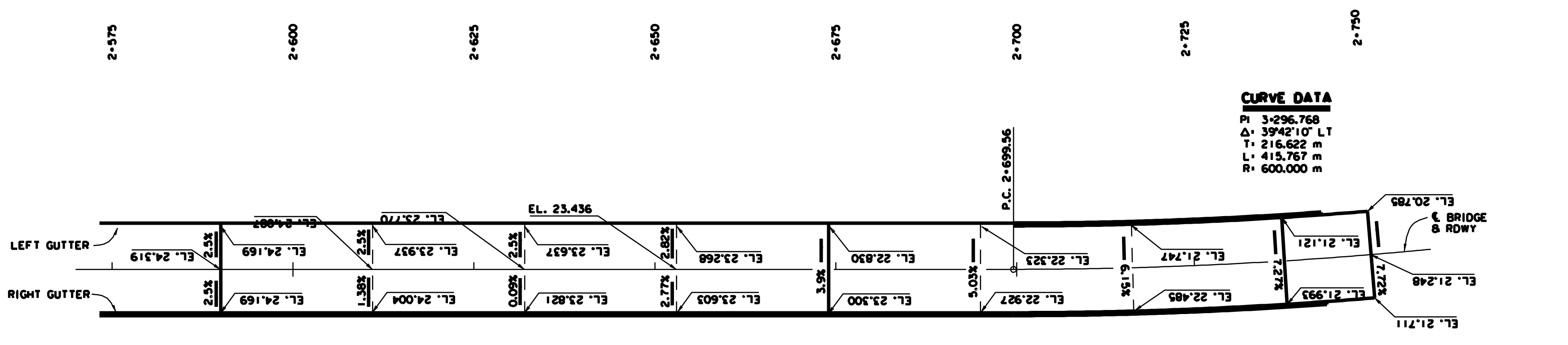
NOTE:
SUPERELEVATION DIAGRAM SHOWN IS USED FOR THE COMPUTATION OF VERTICAL CORRECTIONS ONLY. ACTUAL ELEVATIONS ARE COMPUTED BY ADDING THE CORRECTION TO THE PROFILE GRADE ELEVATION BASED ON VERTICAL CURVE DATA SHOWN ON THE GENERAL PLAN.

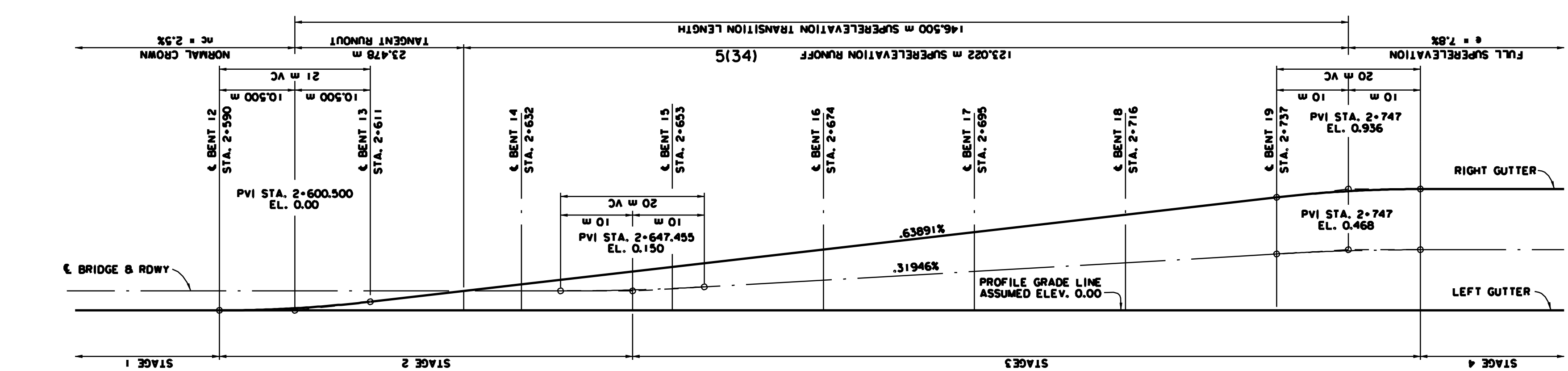
NO.	DATE	REVISION DESCRIPTION	BY

DESIGNED
CHECKED
DETAILED
CHECKED
DATE
SHEET

BRIDGE AND STRUCTURAL DESIGN

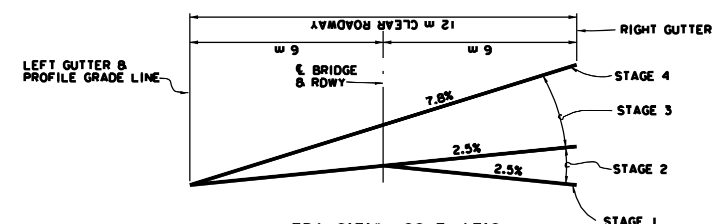
PARISH
FEDERAL PROJECT
STATE PROJECT
SHEET NUMBER





CORRECTION TABLE				
LOCATION	STATION	PROFILE GRADE ELEV.	CORRECTION ℄ BRIDGE	CORRECTION RIGHT GUTTER
BENT 12	2+590	24.169	0.150	0.0
BENT 13	2+611	23.937	0.150	0.067
BENT 14	2+632	23.637	0.150	0.201
BENT 15	2+653	23.268	0.169	0.335
BENT 16	2+674	22.830	0.234	0.470
BENT 17	2+695	22.323	0.302	0.604
BENT 18	2+716	21.747	0.369	0.738
BENT 19	2+737	21.121	0.436	0.872
END APPR.	2+749	20.785	0.463	0.926

NOTE:
SUPER ELEVATION DIAGRAM SHOWN IS USED FOR THE COMPUTATION OF VERTICAL CORRECTIONS ONLY. ACTUAL ELEVATIONS ARE COMPUTED BY ADDING THE CORRECTION TO THE PROFILE GRADE ELEVATION BASED ON VERTICAL CURVE SHOWN ON THE GENERAL PLAN.



TRANSITION SCHEMATIC

NO.	DATE	REVISION DESCRIPTION	BY

DESIGNED
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DATE
SHEET

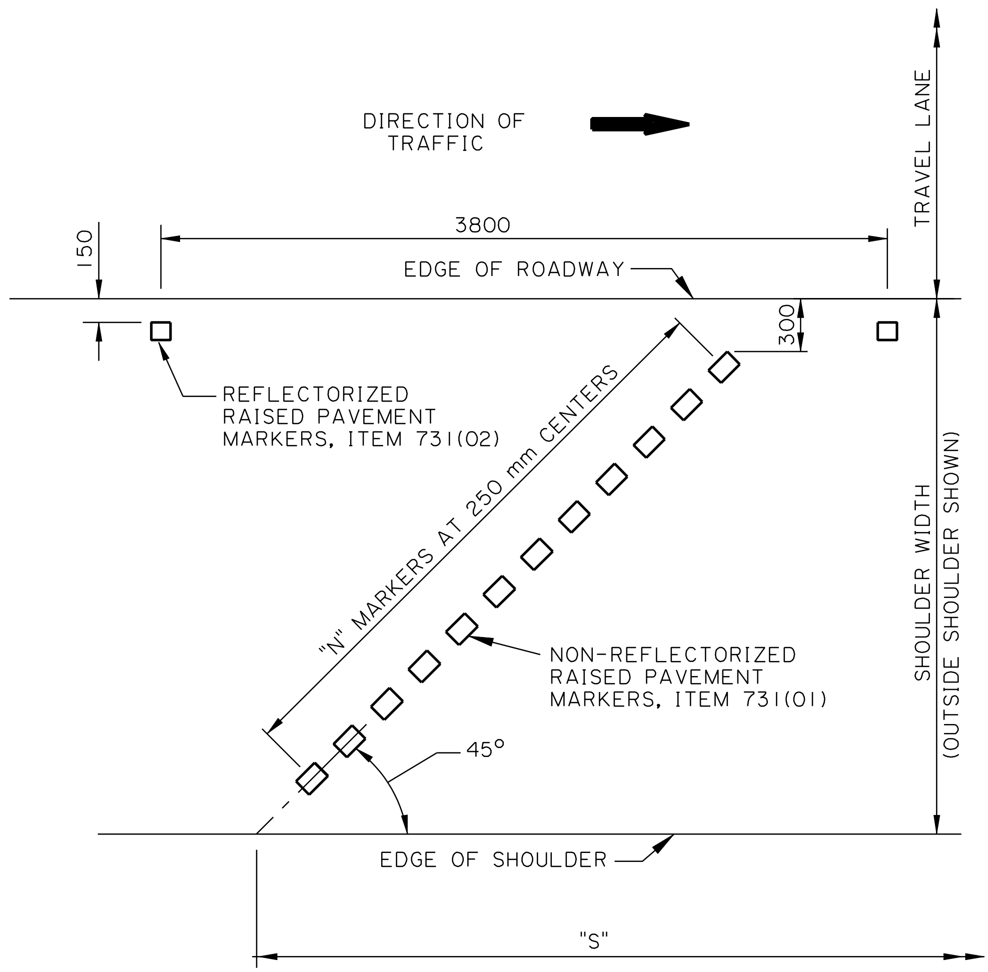
BRIDGE AND STRUCTURAL DESIGN

PARISH
FEDERAL PROJECT
STATE PROJECT
SHEET NUMBER

BRIDGE DECK DRAINAGE

1. On concrete slab span and precast-prestressed concrete girder span bridges, 150 mm diameter deck drains are typically provided along low gutter lines on 3000 mm centers. Spans directly over railroads, roadways, or embankments do not have these drains. On steel bridges, the need for drains is investigated, and when required, drains, which extend their outlet to below the low steel, are used. Design of drains such as scuppers may be found in the Bridge Deck Drainage References, November 1989, DOTD Hydraulics Section and the *Hydraulic Engineering Circular No. 21*, May 1993, Publication No. FHWA-SA-92-010.

2. Structures with significant vertical curves or which incorporate higher embankments (≥3000 mm above natural ground elevation) and have large deck drainage areas (≥250 m^2) are susceptible to embankment erosion and should incorporate bridge end drains where needed.



SHOULDER WIDTH (METERS)	NUMBER OF MARKERS PER DIAGONAL (N)
3.6	18
3.0	15
2.4	11
1.8	7
1.2	3
< 1.2	NOT REQUIRED

* THE DIAGONAL MARKINGS ARE INTENDED FOR TWO-LANE BRIDGES AS WELL AS DIVIDED MULTI-LANE FACILITILES. SEE STANDARD PLAN PM-01-M FOR MORE DETAILS.

BRIDGE LENGTH (METERS)	REQUIRED SPACING (S) (METERS)
> 150	30
60-150	15
< 60	NOT REQUIRED

THE LAYOUT SHOWN BELOW CANNOT BE USED WHERE THE BRIDGE WIDTH IS LESS THAN THE APPROACH WIDTH. SEE PM-01-M FOR DETAILS.

TYPICAL TRAFFIC MARKER PLACEMENT ALONG BRIDGE SHOULDER

DESIGN CRITERIA FOR CONCRETE SLAB SPANS

1. Live load moment shall be based on AASHTO 3.24.3.2 for "E". Both truck and approximate moment shall be calculated. The concrete slab shall be designed for whichever moment is greater.

2. All concrete slab spans with the clear roadway width ≥ 12 m shall be designed for military live load.

3. Load Factor design shall be used in determining the reinforcing steel in the slab to resist the barrier rail design load only when a crash tested model can not be used.

4. Wearing surface, 600 N/m^2

DESIGN CRITERIA FOR CONCRETE BRIDGE DECKS

For the vast majority of girder bridges, the decks are designed and built as reinforced concrete. An alternate, incorporating the use of precast stay-in-place concrete panels, which become composite with a cast-in-place portion of deck, is allowed under some circumstances. Steel stay-in-place forms may be allowed on a case for case basis for use on steel girders only. Loads and stress analysis are as specified by AASHTO and as modified herein.

Analysis

1. The deck is designed as a continuous span over the girders.

2. The Department has chosen to satisfy both working stress and load factor requirements. For working stress design the slab will be designed as doubly reinforced concrete slab with the main reinforcement perpendicular to traffic.

3. The ultimate 28 day compressive strength for the deck concrete (Class AA) shall be **24 MPa** minimum. An allowable stress of **9.0 MPa** shall be used for the working stress method.

4. All steel shall be 420 grade bars.

5. Since the primary stress in the deck is due to live load + impact, the creep factor applied to compression reinforcement shall be neglected.

6. A **600 N/m^2** dead load will be assumed for future wearing surface.

7. Modular Ratio: n = 9 will be used for the design.

8. Reinforcement shall meet the development requirements as stated in AASHTO.

9. The distribution reinforcement indicated in the charts shall be placed in the bottom of the deck.

10. Design section shall equal slab thickness less **15 mm** for section loss due to tire wear.

Deck Design Details

1. Deck thickness shall vary from a minimum of **180 mm** to a maximum of **220 mm** in **20 mm** increments. Optional deck panels will not be allowed as an alternate for **180 mm** decks. Any deck thickness other than 200 and 220 mm, shall be considered a special case, and will have to be approved by the Bridge Design Engineer.

2. A suggested pouring sequence for continuous spans is to be provided for spans over **25 m** in length, giving the minimum rate of pour in cubic meters per hour. The necessary information should be added to the "Miscellaneous Span and Girder Details" sheet 1 of 3. The pouring sequence is based on a 4 hour set time and attempts to minimize cracks in the top of the deck. Try to break the deck into segments at contraflexure points and pour positive moment areas first unless a continuous pour across the support is possible. See Louisiana Standard Specifications article 805.03(d) and limit rate to 45 m^3 per hour.

3. Reinforcing steel shall have **50 mm** cover at the top of the slab, and **25 mm** cover at the bottom of the slab.

4. Main reinforcing bars shall be **#15, or #20** and be placed as near perpendicular to the girders as possible.

5. Longitudinal reinforcing bars shall be **#10,** unless a larger size is needed for continuity over the bents. The top plane of longitudinal steel shall have a maximum spacing of **300 mm** center to center.

6. All bars greater than **#10** will have a detailed maximum length of 18.0 m unless spliced. **#10** bars shall be limited to 12.2 m in length for handling purposes.

7. Main reinforcing steel shall have a minimum spacing of **120 mm** and not greater than the gross deck thickness plus 5 mm.

8. Interpolation of reinforcing steel in deck design table will be allowed only between two sets of identical bar size.

9. 150 mm diameter drains should not be used directly above lower travel lanes, R.R. tracks or abutment slopes, even if revetment is present.

10. Optional deck panels are restricted from use in areas with severely skewed joints (see optional deck panel sheets for geometric limits). On bridges in curves or variable width roadways, the contractor may be allowed to use panels if he provides an independent check of his design and review of all shop drawings at no additional cost.

11. When the use of stay-in-place concrete panels will be allowed, the standard detail sheets will be incorporated into the plans and the general note sheet shall include the item "Optional Deck Details: Precast-prestressed concrete panels conforming to the optional deck detail sheets may be used at the contractor's option."

12. Stay-in-place steel forms will be allowed as an option to the contractor for deep widely spaced plate girders or box girders. The steel panels shall be galvanized in accordance to ASTM A 653M, Z 900 (450 g/m² coverage or 64 µm thick each face) and not increase the dead load from the deck concrete.

13. In order to combat the corrosive effects of salting on primary routes in Districts 04 and 05, as listed below, epoxy coated reinforcing steel shall be required as follows:
    a) All superstructure reinforcing steel, top and bottom mattes.

    b) All reinforcing steel used in bridge barrier rails and bridge sidewalks.

    In addition, the engineer should contact District 04 or 05 to verify if deicing is practiced on a particular structure in an unlisted control section. Galvanized reinforcing steel may be substituted wherever epoxy coated reinforcing steel is required.

    Control Sections in Districts 04 and 05 requiring epoxy coated concrete reinforcing steel:

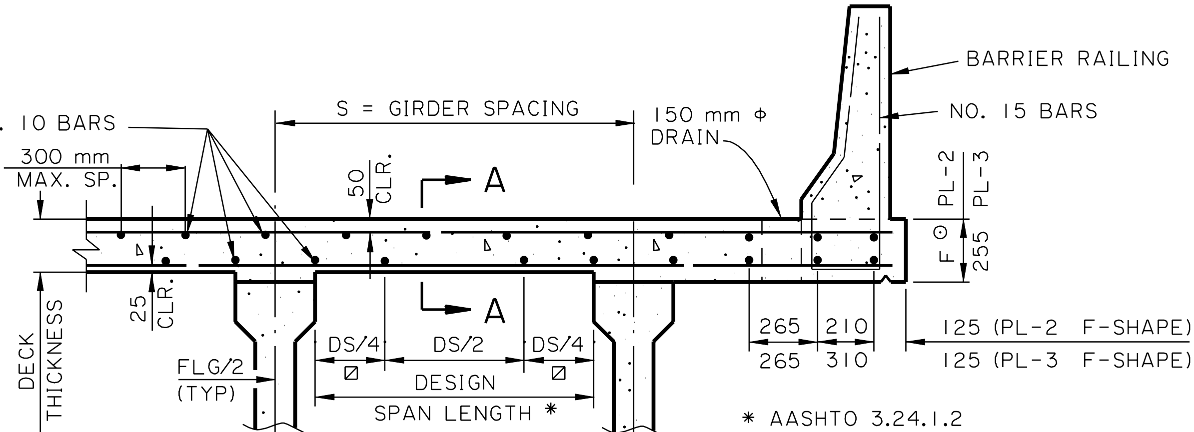
1-01 to 1-09	37-01 to 37-04	83-01 to 83-06
2-01 to 2-06	38-03 & 38-04 & 38-30	85-07
10-02 to 10-03	43-01 to 43-06	86-01 & 86-02
10-05, 10-06 & 10-33	44-01 to 44-03	87-02
11-01 to 11-04	45-01 & 45-03 & 45-30	98-02
15-08 & 15-31	48-01	124-03
16-01 to 16-05	49-01	156-01 to 156-03
20-06 to 20-09	51-04 to 51-08	420-01
21-01 to 21-05 & 21-30	53-06 to 53-09	427-01
23-06 & 23-09 to 23-11	67-07 to 67-09	451-01 to 451-08
25-05 to 25-08	69-02 to 69-04	451-30 & 451-31
26-08 to 26-10	70-01 to 70-07	455-07 & 455-08
27-01 to 27-06	72-01 to 72-02	809-08

    The term control section refers to a section of highway and is designated by the first two digit groupings of a construction project number, for example:

    Project No. 156-02-0053 ----------------------Control Section 156-02

14. Tension development length modification factors for epoxy reinforcing steel must be used. See AASHTO 8.25.2.3

15. When epoxy reinforcing steel is specified, separate quantities will be computed for the epoxy and non-epoxy reinforcing steel, and a separate bid item shall be included in the plans for the epoxy coated reinforcing steel.



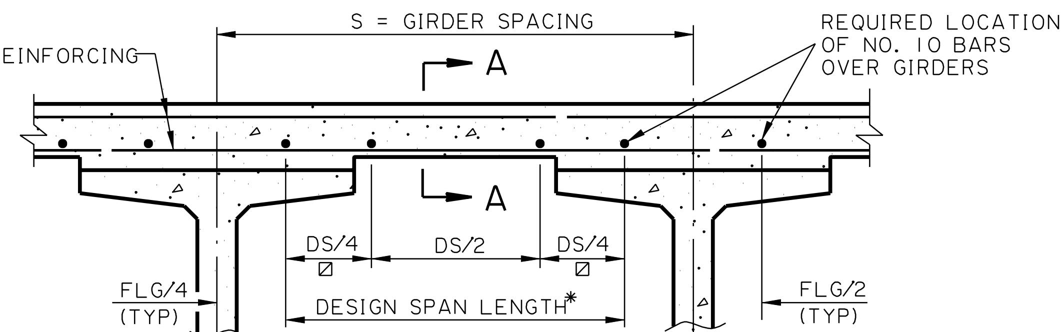
TYPICAL DECK SECTION

(PRECAST PRESTRESSED BEAM)

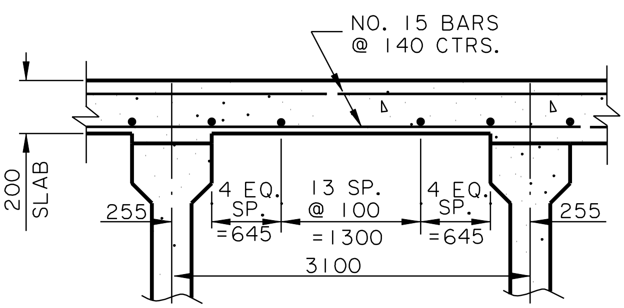
* AASHTO 3.24.1.2

⊙ MINIMUM "F" (PL-2) EQUALS DECK THICKNESS PLUS 20 mm (PLUS 25 mm FOR 180 mm DECK THICKNESS). WHERE DECK THICKNESS VARIES FROM SPAN TO SPAN, SET DIMENSION "F" TO BE CONSISTANT THROUGHOUT THE BRIDGE.

⊠ NUMBER OF BARS WITHIN DS/4 SHALL BE EQUAL TO 1/4 THE NUMBER OF BARS REQUIRED WITHIN DS/2 FROM THE TABLE



TYPE BT DECK SECTION

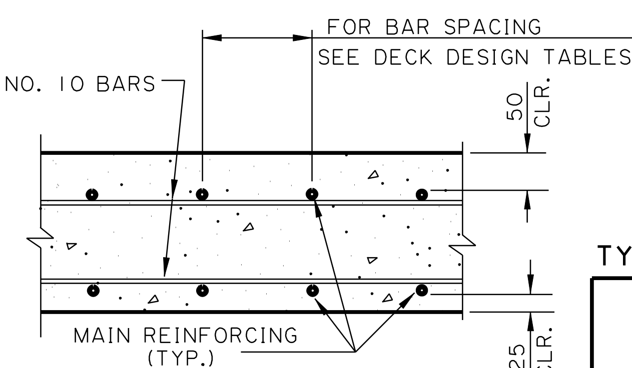


EXAMPLE SOLUTION

EXAMPLE:

GIVEN:
GIRDER SPACING=3100; TYPE IV (b_f = 510 mm)
DESIGN SPAN=2590

SOLUTION:
FROM DECK DESIGN TABLES, THE MOST ECONOMICAL THICKNESS AND REINFORCEMENT COMBINATION IS A 200 mm DECK WITH STRAIGNT NO. 15 BARS AT 140 mm BAR SPACING. USE 13 NO. 10 BARS IN BOTTOM MIDDLE HALF OF SPAN AND THEREFORE 4 NO. 10 BARS IN EACH OUTER QUARTER.



SECTION A-A

TYPICAL SECTIONS

DECK DESIGN

Deck Design Table (hard metric conversion)

Straight reinforcing steel (400 MPa), AA concrete (24MPa)

Slab thickness (mm)	Maximum Design Span (mm)	Main Reinforcement		No. of #10 bars in bottom mid half of span
		Bar No.	Bar spacing (mm)	
	2663	20	140	20
	2465	20	160	16
	2426	**15**	**120**	**14**
180 mm♦	**2304**	**20**	**180**	**13**
	2219	**15**	**140**	**11**
	1938	**15**	**160**	**9**
	1706	**15**	**180**	**7**
	3813	20	120	30
	3505	20	140	25
	3261	**20**	**160**	**21**
200 mm	**3034**	**15**	**120**	**17**
	2708	**20**	**200**	**14**
	2621	**15**	**140**	**13**
	2297	15	160	10
	2035	15	180	8
	1818	15	200	7
	4675	**20**	**120**	**33**
	4224	**20**	**140**	**27**
	3768	**20**	**160**	**23**
220 mm	**3397**	**20**	**180**	**19**
	3088	20	200	16
	2989	15	140	15
	2826	20	220	13

1. **Slab thickness shown above includes 15 mm of wearing surface**
2. **Bar spacing is measured center to center**
3. **Minimum main bar spacing shall be 120 mm.**
4. **Design Load includes a future wearing surface of 600 N/m2 ≈ [12 psf]**
5. **Design spans that are bold type are preferred for a given slab thickness and should provide the most economical design.** Others may be used by the designer if special conditions warrant such use.
6. **Bars in the bottom outer fourth of span = bars in mid half/4.**

♦ Requires approval from the Bridge Design Engineer, see item 1 under Deck Design Details.

Deck Design Table (soft metric conversion) Straight reinforcing steel (420 MPa), AA concrete (24MPa)				
Slab thickness (mm)	**Maximum Design Span (mm)**	**Main Reinforcement**		**No. of #13 bars in bottom mid half of span**
		Bar No.	**Bar spacing (mm)**	
180 mm [3]	2552	19	145	14
	2369	**19**	**165**	**11**
	2218	**19**	**185**	**9**
	2167	**16**	**145**	**8**
	1931	**16**	**165**	**7**
	1705	**16**	**185**	**5**
200 mm	3649	19	125	21
	3366	19	145	17
	3138	**19**	**165**	**15**
	2881	**19**	**185**	**12**
	2613	**19**	**205**	**10**
	2600	**16**	**145**	**10**
	2288	16	165	8
	2033	16	185	6
	1822	16	205	5
220 mm	**4482**	**19**	**125**	**23**
	4057	**19**	**145**	**19**
	3628	**19**	**165**	**16**
	3276	**19**	**185**	**14**
	2982	19	205	11
	2966	16	145	11
	2733	19	225	9

1. **Slab thickness shown above includes 15 mm of wearing surface**
2. **Bar spacing is measured center to center**
3. **Minimum main bar spacing shall be 120 mm.**
4. **Design Load includes a future wearing surface of 600 N/m² ≈ [12 psf]**
5. **Design spans that are shaded are preferred for a given slab thickness and should provide the most economical design. Others may be used by the designer if special conditions warrant such use.**
6. **Bars in the bottom outer fourth of span = bars in mid half/4.**

[3] Requires approval from the Bridge Design Engineer, see item 1 under Deck Design Details.

DESIGN CRITERIA FOR DECKS OF MOVABLE BRIDGES

Vertical Lift Spans

1. Span ≤ 30 m: Use 165 mm concrete deck

2. Span > 30 m: An economic determination shall be made between a grid floor half filled and a 165 mm solid concrete deck. This study shall include the additional machinery and hardware requirements necessary to lift the bridge.

3. Spans, in which ADT requires an extra heavy grid floor, the economic investigation shall be made with a regular steel grid half filled with concrete above.

Swing Spans

1. In general an open steel grid floor shall be used, however, half filled with concrete and a larger counterweight must be investigated.

2. Pivot castings shall be standardized for short, medium, and long spans such that the molds can be reused.

STEEL GRID FLOORS

Steel grid floors are made up of steel plates (bars) or special rolled shapes welded together to form an open grid. The plates subject to main flexural stresses are referred to as bearing bars. The plates perpendicular to the bearing bars provide lateral support and distribution of the load to the bearing bars and the shallower bars between and parallel to the bearing bars provide more uniform surface respectively. Loads and stress analysis are as specified by *AASHTO*.

Commentary

Steel grid floors are generally used in movable bridges to minimize the weight of the movable span's deck. This in turn reduces the load requirements on the machinery required to perform the movement operations.

Commercial types of open grid flooring are fabricated from plates or special rolled shapes, and should conform to Standard Plan GF-1 or GF-2. One of these open grid standards shall be used on all grid-flooring applications.

Analysis

1. 1. Grid flooring shall be designed as continuous over stringers or girders parallel to the centerline of the roadway.

2. Use heavy-duty grid floor when ADT is high (ADT > 7000).

3. Use half-filled grid floor in the first bay on the long cantilever of swing span bridges to protect machinery below if weight is critical.

4. Grid floors are available in ASTM A 709 Grade 250, 345, and 345 W steels. The design span will determine the type used.

Design Details

1. The merits of alternate open grid floor systems should be investigated in terms of the basic AASHTO Specifications.

2. Bearing bars shall be perpendicular to the stringers or girders and welded to the top flange of the same at each juncture.

3. If two or more coats of paint are required on the grid floor, the cost to galvanize is more economical and should specified.

4. For 165 mm concrete decks, incorporate 38 mm clear cover for reinforcing steel, top and bottom

DECK JOINT

The discussion herein will pertain to deck joints for girder span bridges, as slab span joints are in accordance with the standard plans for slab span bridges.

For prestressed girder spans there are two types of joints currently being used; open joints and strip seal joints. Open joints are used at intermediate bents in rural areas and for stream crossings in urban areas where aesthetics is not as critical. For urban overpasses and interchanges exposed to the public view, strip seal joints are used to prevent unsightly staining and debris accumulation from drainage effluent. Strip seals are used at all end bents to prevent erosion. Both strip seal and open joints are capable of handling the expansion of up to **80 m** of prestressed girder span (**75 mm** maximum opening). For a single continuous span unit in which all of the expansion occurs at the abutments, the distance between joints could theoretically be doubled, however, bent restraint must be taken into account at continuity bents with span fixity.

All steel girder spans will have sealed joints. Strip seal joints are capable of handling the expansion of up to **56 m** of steel girder span. (A 112 m continuous unit can be handled if abutments are at each end with the unit fixed at mid-point.) For longer spans, finger joints will generally be employed with a trough (at 8% minimum slope) provided to divert the drainage away from the steel superstructure. Generally thick finger plates without stiffeners are more desirable than thinner plates with stiffeners, as the thick plates add more inertia to the joint as well as provide a better detail for fatigue resistance. For finger joints on curved girder spans, the designer is advised to refer to the AASHTO's "Tentative Design Specifications for Horizontally Curved Highway Bridges" regarding the orientation of the fingers and the bearings. Joints shall be furnished in one piece without butt welds unless it is impractical due to plate length availability, in which case only one shop butt weld will be permitted.

The Bridge Design Engineer must approve the use of prefabricated or modular expansion joints in lieu of finger joints.

The use of steel reinforced elastomeric joint seals is prohibited.

In gore areas of new construction or in severely skewed spans, portions of the joint may have severe kinks and the designer is advised to confer with the joint manufacturer to insure a proper fit of the strip seal.

Open joints shall be in accordance with the "Miscellaneous Span and Girder Details", and strip seals shall be in accordance with the "Strip Seal Joint Details". The design of the gap setting of open joints is similar to strip seals.

Assume the normal installation temperature for finger joints is 20°C. The project engineer should adjust the opening when the temperature in the structure differs from normal by more than 8°C. The design should use a factor of safety of 2 for overlap and for opening.

Design Criteria for Strip Seals

1. Span length "L" equals the expansion distance. In some instances it will equal the length of the particular continuous span unit in question. In other cases it will equal the distance between the assumed points of fixity of two consecutive continuous span units.

2. TEMPERATURE RANGE (°**C**)

TEMPERATURE RANGE (°C)	RISE	FALL
Concrete Girders	17°	22°
Steel Girders	34°	34°

COEFFICIENT OF THERMAL (°C)
EXPANSION

CONCRETE	= 0.0000108
STEEL	= 0.0000117

3. SHRINKAGE AND PRESTRESS CREEP

20.833 mm per **100 m** for PRESTRESSED GIRDERS
10.417 mm per **100 m** for STEEL GIRDERS

4. JOINT OPENING | MINIMUM OPENING | MAXIMUM OPENING

JOINT OPENING	MINIMUM OPENING	MAXIMUM OPENING
MAX. TEMPERATURE	**25 mm**	
MIN. TEMPERATURE		**75 mm**

(The **25 mm** criteria may be violated if long term shrinkage and creep are considered.)

5. Generally, only **100 mm** (4 inch) strip seal glands will be used in all strip seal joints, regardless of the actual movement. Therefore, strip seal design will simply be a matter of setting the joint opening at installation temperature, using the criteria listed above.

Design Example: Prestressed Concrete Girder Spans

A series of 4-span continuous units, with 20 m spans

1. **L = 80 m**

2. Thermal movement

   17°C rise (expansion)
   $= 17 \times (0.0000108) \times L$
   $= 0.00018 \times L$
   $= 14.4$ mm

   22° C fall (contraction)
   $= 22 \times (0.0000108) \times L$
   $= 0.00024 \times L$
   $= 19.2$ mm

3. Installation dimension

   25.0 mm (min.) + thermal expansion
   = 25.0 mm + 14.4
   = 39.0 mm

4. Creep and shrinkage

   $= (20.833 \text{ mm} \div 100 \text{ m})\ (\text{m} \div 1000 \text{ mm}) \times L$
   $= 0.00021 \times L$
   $= 16.7$ mm

5. Max. opening < 75 mm

   = installed dimension + [creep and shrinkage + thermal contraction]
   = 39.0 mm + 16.7 mm + 19.2 mm

$= 74.9 \text{ mm} < 75 \text{ mm}$
$\therefore$ O.k.

EQUATION FOR CHECKING THE MAXIMUM JOINT OPENING

$$\Delta_{(max)} = (0.00018 \times L) + [(0.00024 \times L) + (0.00021 \times L)] < [75 \text{ mm} - 25 \text{ mm}]$$
$$= 0.00063 \times L < 50 \text{ mm}$$
$$L < 80 \text{ m concrete}$$

Design Example: Steel Girder Spans

A series of 50 m simple spans

1. $L = 50 \text{ m}$

2. Thermal movement
   34°C rise (expansion)
   $= 34*(0.0000117) \times L$
   $= 0.0003978 \times L$
   $= 19.9 \text{ mm}$
   34°C fall (contraction)
   $= 19.9 \text{ mm}$

3. Installation dimension
   25 mm (min.) + thermal expansion
   $= 25 \text{ mm} + 19.9 \text{ mm}$
   $= 45.0 \text{ mm}$

4. Deck shrinkage
   $= (10.417 \text{ mm} \div 100\text{m}) \times (\text{m} \div 1000\text{mm}) \times L$
   $= 0.0001042 \times L$
   $= 5.2 \text{ mm}$

5. Max opening $\leq$ 75 mm
   = installed dimension + [deck shrinkage + thermal contraction]
   $= 45.0 \text{ mm} + 5.2 \text{ mm} + 19.9 \text{ mm}$
   $= 70.1 \text{ mm} < 75 \text{ mm}$
   $\therefore$ O.k.

EQUATION FOR CHECKING THE MAXIMUM JOINT OPENING

$$\Delta_{(max)} = 0.0003978 \times L + [(0.0003978 \times L) + (0.0001042 \times L)] < [75 - 25 \text{ mm}]$$
$$= 0.0009 \times L < 50 \text{ mm}$$
$$L < 56 \text{ m}$$